RESERVOIRS AND TANKS

G. P. MANNING

U d'/of OTTAWA
39003006493075

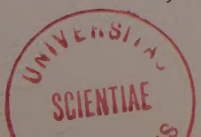
CONCRETE RESERVOIRS AND TANKS

BY

G. P. MANNING

M. Eng., M. Cons. E., M.I.C.E.

CONCRETE PUBLICATIONS LIMITED
60 BUCKINGHAM GATE, LONDON, S.W.1



First published in this form . . . 1967

84.68

TD
395
.M25
1967

Made and Printed in Great Britain by Butler & Tanner Ltd., Frome and London

CONTENTS

PAGE

PREFACE v

NOTATION vi

CHAPTER 1.—PRELIMINARY ARRANGEMENT, DIMENSIONS AND BASIC DESIGN . . 1
Cover to reinforcement. Effects of earth banking. Cement. Mixes. Watertightness.

CHAPTER 2.—CIRCULAR TANKS WITH SLIDING JOINT AT BASE OF WALL . . . 8
General design principles. Arrangement of reinforcement. Early tanks. Accidental overload. Shrinkage and temperature stresses. Sliding joints. Tank Floors. Economical dimensions. Semicircular tanks.

CHAPTER 3.—RESTRAINT IN OPEN-TOPPED CIRCULAR TANKS WITH WALLS OF UNIFORM THICKNESS 21
Assumptions. Fundamental equations. Shape of wall. Wall thickness in relation to CP. 2007. Partial restraint. Loss of angular restraint.

CHAPTER 4.—TOP RESTRAINT IN CIRCULAR TANKS 44
General design principles. Practical application.

CHAPTER 5.—RESTRAINT IN OPEN-TOPPED CIRCULAR TANKS WITH TAPERED WALLS 49
Approximate analysis. Partial restraint.

CHAPTER 6.—RESTRAINT IN LARGE-DIAMETER SHALLOW CIRCULAR TANKS . . 58
General design principles. Movement of base. Interrupted circular walls.

CHAPTER 7.—RESTRAINT IN LARGE SHALLOW RECTANGULAR TANKS . . . 71
General design principles.

CHAPTER 8—MASS CONCRETE WALLS 74
Site conditions. Unbanked gravity walls on dry, drained and wet sites. Banked walls. Footing slabs. Counterfort walls.

CHAPTER 9.—CANTILEVERED SLAB WALLS 82
Theoretical proportions. Thickness of stalk. Example of preliminary design for cantilevered slab wall and for base. Example of preliminary design for division wall. Stability. Cost. Expansion joints.

CHAPTER 10.—CHANNELS WITH SLOPING BOTTOMS 101
Theoretical analysis for differing conditions.

CHAPTER 11.—CANTILEVERED COUNTERFORT WALLS 107
General design. Wall slab. General proportions. Stability. Base slab. Back beam. Counterforts. Secondary effects. Constructional details. Sloping and other shapes of wall.

CHAPTER 12.—LARGE RESERVOIRS WITH WALLS SPANNING VERTICALLY . . . 135
Liquid and earth loading. Uniform slabs. Tapering slabs. Longitudinal movement. Slab-and-beam walls.

CHAPTER 13.—NARROW TANKS WITH WALLS SPANNING VERTICALLY . . . 145
General analysis. Culverts. Suspended flumes. Mesh-panel walls.

CHAPTER 14.—USE OF SEGMENTAL ARCHES IN RESERVOIR WALLS 161
General analysis and design.

CHAPTER 15.—SPLAYED FLOOR AND WALL SLABS 169
General analysis and design.

CHAPTER 16.—RESERVOIR FLOORS 173
Supported floors on good, patchy or compressible ground. Two- and three-layer floors. Linings. Sumps. Suspended floors. Floors spanning one- and two-ways between sleeper walls. Floors supported on piers. Floors supported on piles. Drainage.

PAGE

CHAPTER 17.—RESERVOIR AND TANK WALLS SUPPORTED ON PILES 196
Wall with slip joint. Restrained walls. Cantilevered walls.

CHAPTER 18.—ELEVATED TANKS 209
Early designs. Recent designs. Columns. Elementary flat-slab floors. Example of preliminary design for circular tank. Practical considerations.

CHAPTER 19.—DEEP RECTANGULAR TANKS 221
General analysis.

CHAPTER 20.—TANKS BELOW GROUND WATER LEVEL 225
General design principles. Loading on walls and on floor. Design of walls. Earth-loaded tanks.

CHAPTER 21.—PYRAMIDAL AND CONICAL TANKS 236
Tanks on hard dry soil. Tanks on soft wet ground. Flotation. Construction in soft wet ground. Imhoff tanks.

CHAPTER 22.—SWIMMING POOLS 249
Small private pools. Larger pools. General design principles.

CHAPTER 23.—RESERVOIR ROOFS. 258
Slab-and-beam roofs. Mesh-panel roofs. Sunk-panel flat-slab roofs. Elementary flat-slab roofs. Edge details. Expansion joints in roofs. Small circular roofs.

CHAPTER 24.—MINING SUBSIDENCE 271
General principles. Prestressed concrete tanks.

CHAPTER 25.—CIRCULAR TANKS UNDER LATERAL LOADING 279
General design principles. Wind on buried tanks. Tanks subjected to seismic effects. Wind effects on a tank for a spirally-guided gasholder. Forces due to overturning. Forces due to shear. Thrust on rollers. Wind effects on large structures. Gasholder tanks. Examples.

CHAPTER 26.—NOTES ON CONSTRUCTION 303
Storage of industrial liquids. Storage of hot liquids. Storage of very cold liquids. Concrete materials and placing. Formwork. Settlement tanks.

CHAPTER 27.—EXAMPLES OF COMPLETED WORK 321

CHAPTER 28.—RESTRAINT IN PRESTRESSED CIRCULAR TANKS 350
Wall of uniform thickness with slip joint. Uniform wall fully restrained at base. Uniform wall hinged at base. Stability of thin circular wall construction and useful life of prestressed tanks.

CHAPTER 29.—REPAIRS 361
General. Cracks in walls of reservoirs above ground. Cracks in walls of underground tanks. Repairs to floors. General repairs to walls. Repairs to roofs.

APPENDIX 1.—SOME NOTES ON BRITISH STANDARD CODE OF PRACTICE CP. 2007:1960 365

APPENDIX 2.—WORKING STRESSES AND SPECIFICATIONS 375

APPENDIX 3.—DR. REISSNER'S METHOD OF CALCULATING RESTRAINT . . . 377

ACKNOWLEDGEMENTS 382

INDEX 383

PREFACE

This book is a successor to the original work on the same subject by the late W. S. Gray, B.A., M.A.I. (Dublin), A.M.I.C.E. The text has been almost entirely re-written and considerably extended.

Unless specifically stated to the contrary the text refers to unlined tanks to contain water at or near air temperature. Tanks to contain hot, very cold, corrosive or highly penetrating liquids require special treatment and special lining (see Chapter 26).

It is assumed that the reader has a working knowledge of the basic principles of reinforced concrete design and the theory of structures.

G. P. M.

NOTATION

THE notation adopted is generally in accordance with that used in CP. 114:1957, the most commonly-used symbols being as follows :

a : overall depth of column section.
A_{sc} : cross-sectional area of steel in compression.
A_{st} : cross-sectional area of steel in tension.
b : breadth of rectangular beam or member.
D : diameter of cylindrical tank.
d : overall depth of member (beam section).
d_1 : effective depth to tension reinforcement.
d_n : depth to neutral axis ($= n_1 d_1$).
E_c : elastic modulus of concrete.
E_s : elastic modulus of steel.
f_{bl} : local bond stress.
f_{cb} : actual compressive stress in concrete in bending.
f_{cc} : actual compressive stress in concrete in direct compression.
f_{ct} : actual tensile stress in concrete.
f_{st} : actual tensile stress in steel.
H : total height or depth of tank.
h : head of water.
K : stiffness factor ($= I/L$).
l_a : lever arm.
m : modular ratio.
p : intensity of water pressure.
p_{cb} : maximum permissible compressive stress in concrete in bending.
p_{cc} : maximum permissible compressive stress in concrete in direct compression.
p_{ct} : maximum permissible tensile stress in concrete.
p_{st} : maximum permissible tensile stress in steel.
Q : total shearing force.
q : shearing stress.
R : radius (usually of circular tank).
r_t : proportion of reinforcement in tension ($= A_{st}/bd_1$).
s_b : adhesion or bond stress.
t : overall thickness of member.
T : total tensile force.
W : total load.

CHAPTER ONE

PRELIMINARY ARRANGEMENT, DIMENSIONS AND BASIC DESIGN

In this book the term 'reservoir' is intended to apply generally to service reservoirs similar in size and type to the examples shown in Chapter 27. The term ' tank ' is here generally applied to tanks found in industrial or public utility undertakings such as gas works, sewage works, cement works, etc.

Reservoirs are normally placed on the highest available site. This generally means that the ground is sound and the ground-water level is well below the surface. To construct a reservoir in such circumstances it is often possible simply to excavate a hole some 20 ft. or 30 ft. deep and line it with concrete, as shown in *Fig.* 1.1.

Fig. 1.1.

A 6-in. layer of reinforced concrete laid on 3 inches of mass concrete is all that is necessary, and the arrangement shown in this figure generally provides the cheapest method of water storage. The only difficulty in construction is the top shuttering for the slopes, which must be clearly specified and properly designed to enable the concrete to be soundly consolidated. Unless the site is well above the surrounding country the floor level of a totally excavated reservoir, such as that shown in *Fig.* 1.1, may be too low to produce the necessary head of pressure in the supply system to supply the higher parts of the neighbourhood, but may be sufficiently high to supply all the lower parts of the neighbourhood. There may then be a choice between raising the floor level of the reservoir or leaving the floor level as it is and providing a small water tower to supply the higher parts of the neighbourhood.

If the top water level to be provided is only a few feet above ground level a mass concrete wall, as in *Fig.* 1.2, is probably the best solution, but for greater depths

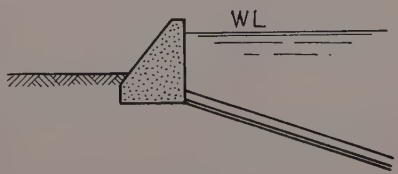


Fig. 1.2.

some form of reinforced concrete wall is generally necessary, as in *Fig.* 1.3. Generally the wall is the most expensive item (more particularly since the publication of Code CP.2007) and the designer should examine all reasonable means of reducing the effective height. If the floor level is at least 2 ft. below the ground it may be cheaper to build up in mass concrete to reduce the effective height of the reinforced

concrete wall, as shown in *Fig.* 1.4. If the floor level is slightly above ground level it may be cheaper to make up in solid mass concrete or in hand-packed stone flushed solid with a sand-and-cement mortar, but when the floor must be several feet above ground level the provision of some type of suspended floor becomes essential (see Chapter 16).

The need for a suspended floor may be more apparent than real. If the level *x–x* in *Fig.* 1.4 represents the lowest usable water level (all water below this level having too low a head of pressure) it may, particularly in a large reservoir, be cheaper to build the reservoir as shown, rather than put in an expensive suspended floor at level *x–x*, and also cheaper than filling solid below level *x–x* with mass

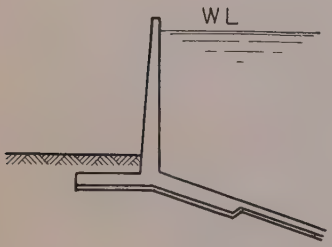


Fig. 1.3.

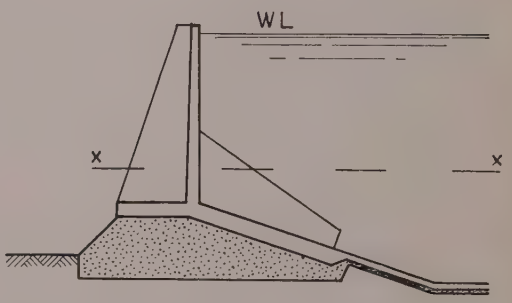


Fig. 1.4.

concrete. In fact this design consists of filling solid below level *x–x* with unusable water. Most modern reservoirs are roofed over and this complicates the question of cost since a reservoir with sloping sides, as that in *Fig.* 1.1, has a larger roof area than one of similar capacity and similar depth but with vertical sides. The determination of the best dimensions for a reservoir to hold a given volume of water with a given minimum pressure head in the main may therefore entail the consideration of several alternative designs for the reservoir itself and possibly several alternative combinations of reservoir and water tower.

Tanks, by contrast, are often sited on low-lying quaternary ground and sinking them below ground level entails heavy construction. Their dimensions and levels are usually fixed by the commercial processes they serve, and the designer has generally no need to consider alternative depths and diameters.

Cover to Reinforcement

The subject of the correct amount of concrete cover necessary to the reinforcement, which is vital in water-containing work, seems to be widely misunderstood. Few people seem to realize that 1 in. of actual cover will certainly protect the steel while 2 in. of specified but non-existent cover will certainly not. Many exposed reinforced concrete structures more than 20 years old show rust marks and some spalling, and this has led to a surprising amount of erroneous thinking. In a small minority of cases of rusting the trouble may be due to the drawings and specification calling for too little cover but in the vast majority of cases the drawings and specification call for ample cover, but they neither show nor specify the means by which this cover is to be maintained, and the contractor has therefore failed to maintain it. Some engineers seem to think that if they vaguely and hopefully specify 2 in. of cover the contractor may reasonably be expected to provide a minimum of 1 in.

Clearly the remedy is not to specify more and more non-existent cover but to insist on the maintenance of the actual cover specified.

Bars in floors may be supported on purpose-made sand-and-cement blocks as shown in *Fig.* 1.5. In walls and columns the bars may also be kept away from the shutters by sand-and-cement blocks but are held much more positively by asbestos-cement spacers of the type shown in *Fig.* 1.6. These spacers are circular

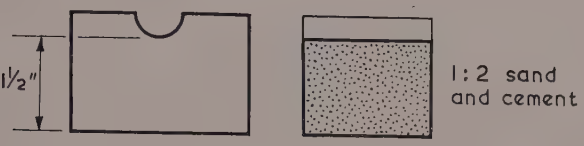


Fig. 1.5.

and each has a central hole $\frac{1}{16}$ in. larger than the diameter of the bar over which it is threaded. They are $\frac{3}{8}$ in. thick and one spacer per square yard of wall shuttering should be sufficient. Means for keeping the steel in place must be shown on the drawings, described in the specification, and included in the bill of quantities.

An actual cover of 1 in. outside column links and small bars and $1\frac{1}{2}$ in. outside main bars up to $1\frac{1}{2}$ in. in diameter is ample, provided that the specification also

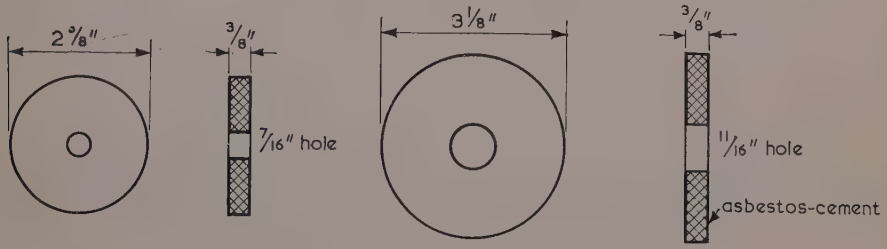


Fig. 1.6.

insists that the ends of the binding wire are turned inward and not allowed to encroach on the cover, as this is a frequent cause of the commencement of rusting.

Since the cement tends to rise to the surface and the steel tends to sink, a nominal cover of 1 in. above the top bars in a beam or a slab is ample. However much honeycombing may occur on the bottom or cheeks of a beam (or on the face of a wall) the top surface of a beam or a slab is always sound and solid. The reinforcement may be displaced sideways or downward by ramming or vibrating, and such displacement is not visible until it is too late, but it can scarcely be displaced upwards as the top bars remain in sight until the concreting is almost complete.

Effects of Earth Banking on Calculated Working Stresses

Many old service reservoirs have vertical brick walls which appear, at first sight, to be retaining walls. The construction, however, consists essentially of a circular hole with a lining of puddle clay which has a facing of brickwork in lime mortar. The water pressure passes through the brickwork facing, on to the puddle clay and thence on to the surrounding ground.

In the design of a modern reservoir with vertical walls it is the usual practice

to make the walls themselves sufficiently strong to carry all the water pressure, and any support that the walls may receive from the external earth banking is ignored.

It seems irritating and illogical that ' old-fashioned ' design can shed 100 per cent. of the water pressure on to the earth banking while ' modern ' design can shed none, particularly since the wall must also be made sufficiently strong to resist heavy inward earth pressure when the reservoir is empty. Surely, it is often argued, some ' safe ' fraction of this earth pressure can be set off against the water pressure. Even if the specification insists that the reservoir is filled and tested before any backfilling is commenced, some will still argue that this is only a ' temporary overload ' justifying an increase of 25 per cent. in the working stresses. The only type of backfill that can be relied on to produce active earth pressure is clean dry sand under desert conditions. Reinforced concrete construction is much too rigid to develop counterpressure from any soil softer than solid chalk under normal seasonal variations.

In general, then, the reinforced concrete walls must be designed to carry the whole of the water pressure at normal working stresses.

Cement

Before 1914, that is before the introduction of rapid-hardening cement, long structures were built without expansion joints, yet developed no major cracks and few, if any, minor ones. This was before the term ' water-cement ratio ' was coined and mixes were usually wet compared with modern practice. The mild steel reinforcement and the concrete aggregates were exactly the same as now, the only difference being in the type of cement used.

Provided that a structure is reasonably well designed and reasonably well built the only danger to be feared is the development of shrinkage and temperature cracks. A well thought-out programme of concreting and careful curing of newly-placed concrete will reduce the danger of such cracks occurring, but every effort should be made to secure the use of a cement that generates a minimum of heat (and generates this slowly), and that develops a reasonably high tensile strength with high plastic yield in tension. Generally speaking, rapid-hardening cement should be avoided. According to technical reports some cements sold as ' normal ' are more rapid-hardening than others sold as ' rapid-hardening ' and the actual properties of the cement should be ascertained before use. This raises an important question: when cement is required for watertight work, should it have a much higher minimum tensile strength than that specified in B.S. 12 and should there be a minimum specified strain on the sand-and-cement standard specimen under prolonged tensile loading?

For the results of experiments on the tensile strain of concrete the reader is referred to work described elsewhere.* In seeking to interpret these results it must be remembered that the test specimens were small and were made with a Japanese cement.

Concrete Mix

When selecting a suitable concrete mix it should be remembered that high compressive strength is seldom if ever required in watertight work (other than

* Ohno and Shibata.—Report in RILEM Bulletin No. 4 (New Series) October 1959. Pp. 24 to 31.

prestressed concrete construction) but high tensile strength, high adhesion and non-porosity are always required. This means that the whole emphasis should be on workability and not on test-cube strength. For this reason the author never specifies anything weaker than a nominal 1 : 1½ : 3 mix and would consider 1 : 1¼ : 2½ for a very small tank. Specification of the water-cement ratio or slump test results should be postponed until specimen batches of concrete have been made and placed.

Watertightness

Concrete made with rounded shingle aggregate is easier to make watertight than a similar mix made with crushed stone aggregate. A series of similar tanks were all built on one site by one contractor using the same general foreman. All the tanks except one were built with rounded shingle aggregate. The tank in which crushed granite aggregate was used developed a number of small leaks, but no corresponding leaks developed in any of the other tanks.

It is a matter of opinion whether the addition of waterproofers, plasticizers, air-entraining agents, and the like, really improves the concrete as the use of such products may produce a sense of false security and increasing carelessness. No additive can compensate for shoddy workmanship.

First Principles of Watertight Design

Before any reinforced concrete structure will function satisfactorily at least three major considerations must be satisfied.

1. The stresses in the concrete must be kept within reasonable limits.
2. The stresses in the steel must be kept within reasonable limits.
3. The adhesion or anchorage between the concrete and the steel must be adequate.

In all but the smallest structures there is a further consideration.

4. Reasonable attention must be given to shrinkage and temperature stressess

When dealing with unlined water-retaining structures all these considerations assume a special significance in relation to the production of watertight work. The first and second are now generally understood although there is no general agreement as to the permissible limits of the tensile stresses in concrete or steel.

Generally speaking the third consideration is not fully understood and Codes of Practice do not go into the matter in sufficient detail. In a building frame or similar structure the tensile stresses are normally due solely to bending moments, but tanks, in general, are subject to additional tensile stresses due to the bursting action of the liquid they contain; increased anchorage is therefore required.

Very few designers give enough thought to the effects of shrinkage and temperature stresses and the recommendations given in Codes of Practice are not very helpful in dealing with such stresses.

In a normal reinforced concrete building the beams are continuous and are relatively stronger and sounder at midspan than over the supports. The principle of greatest strength allows moment to be transferred from the support section to midspan at the risk of causing fine hair cracks to occur over the supports. The fact that these cracks may be reduced in size and number by substituting deformed

bars for plain round bars indicates that they are not solely due to high tensile stresses in the concrete but are due partly, at any rate, to local failure of adhesion.

In the normal building the top bars in the beams over the points of support may be taken past the point of contraflexure but in watertight work, particularly in circular tanks, the bars may have to be terminated where there are still considerable tensile stresses, thus causing high local adhesion stresses near the ends of the

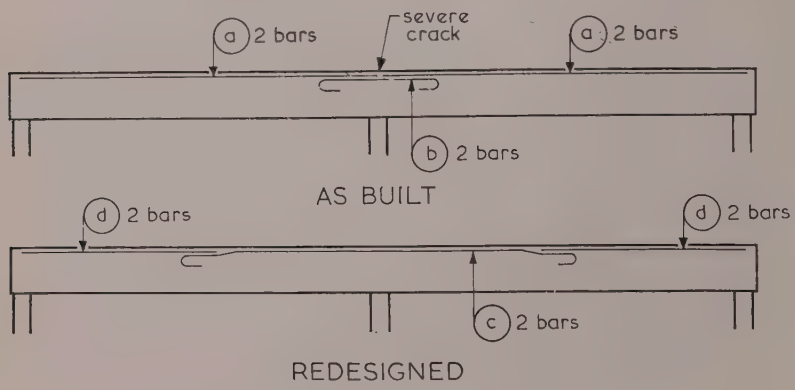


Fig. 1.7.

bars. All main bars in tension must have end hooks and ample laps, and the positions of the end hooks must be spread over as large a distance as possible. Two examples based on practice are shown in *Figs.* 1.7 and 1.8.

A deep narrow beam extending over two spans and carrying the side of a feeder channel is shown in *Fig.* 1.7. The two bars *b* were ample to resist the reversed moment over the central support, but the design provided two heavy bars *a* in the

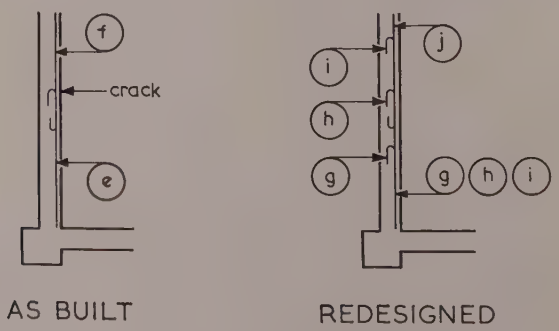


Fig. 1.8.

top of each span which were stopped short about one inch away from the centre-line of the middle support. A severe crack opened between the ends of these bars: had the design been revised as shown, it is most unlikely that any such crack would have developed.

A vertical section through the wall of a small water tank in which the bottom was suspended from the walls, is shown in *Fig.* 1.8. The bars *e* were all carried to the points indicated by theory and were all stopped at this point. Bars *f* had the minimum overlap with bars *e*. The wall cracked along the line of the top

of the hooks on bars *e*. Had these bars been divided into three series *g*, *h* and *i* stopping at different levels, this crack would not have developed.

Shrinkage and temperature stresses may often be avoided by dividing a building into separate sections with open expansion joints between them. This is not always possible with water-retaining structures, but the effects of shrinkage stresses may be reduced by insisting on a concreting programme that reduces these stresses to a reasonable minimum. For example if the floor of a tank is concreted and allowed to harden and dry out in the sun for ten days or more before the walls are commenced then numerous vertical cracks in the base of the wall are inevitable. There should be an interval of not more than two days between concreting a section of the floor and that section of the wall standing on it (unless the two are separated by an efficient sliding joint). With a large reservoir there must be many construction joints where a considerable time interval has elapsed between concreting adjoining sections but these should be located where the structural stresses are a minimum, or at any rate at those positions where any cracks may be easily reached and repaired. Experience shows clearly that there is very much less trouble with vertical construction joints in walls than with horizontal joints. If the designer has to choose, he should always select the former. The design and the specification should indicate the positions of construction joints and insist on a definite order of concreting. It is common practice now to put water-bars (metallic or synthetic) in most, if not all, construction joints. The designer should not allow this to lull him into a false sense of security and should in no wise relax any other reasonable precaution to ensure leak-proof joints.

CHAPTER TWO

CIRCULAR TANKS WITH SLIDING JOINT AT BASE OF WALL

The simplest type of tank is shown in *Fig.* 2.1. This is a circular tank with a wall of uniform thickness t and provided with a frictionless sliding joint at the bottom of the wall (assuming that such a thing exists). (The writer is not in favour of sliding joints, particularly for small tanks, but sometimes they cannot be avoided, and since some experienced engineers favour them, they are therefore discussed here.)

If w is the weight in pounds per cu. ft. of the liquid in the tank, D the diameter

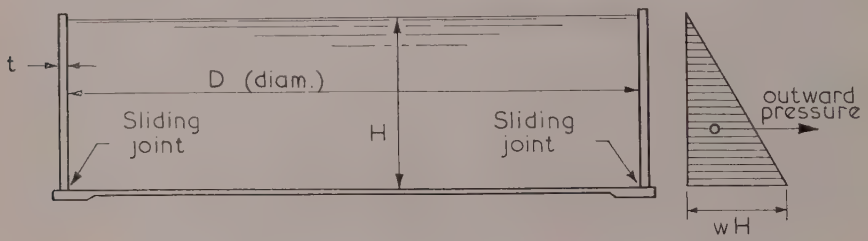


Fig. 2.1.

in feet and H the depth in feet, then the pressure at the base of the wall is wH lb. per sq. ft.

The wall is clearly in a state of simple hoop tension, the amount of which per foot height of wall at the base of the wall is $\frac{1}{2}wHD$ lb.

If t is the thickness of the wall in inches, the maximum tensile stress occurs at the base of the wall and is

$$\frac{wHD}{2} \times \frac{1}{12t} = \frac{wHD}{24t} \text{ lb. per sq. in.}$$

If $D = 50$ ft., $H = 15$ ft. and $t = 6$ in., the maximum tensile stress is $\frac{w \times 15 \times 50}{24 \times 6}$.

If the tank contains water, then $w = 62{\cdot}5$ and the maximum tensile stress

$$= \frac{62{\cdot}5 \times 15 \times 50}{24 \times 6} = 325 \text{ lb. per sq. in.}$$

The tensile stress in the wall obviously varies directly as the depth below the surface. The figure of 325 lb. per sq. in. assumes that the wall is constructed of uniform material. With reinforced concrete this is not so and a complication at once arises since both concrete and steel have to be considered. During the process of first filling the tank it seems likely that the concrete takes the whole of the tension in the initial stages, but as filling proceeds the tension is shared between concrete and steel in the ratio of their elastic moduli. (The value of this ratio is usually taken as 15.) This assumes that each lift of the tank wall is concreted in one continuous operation without any vertical construction joints (either intentional or

accidental due to breakdown of plant). At a vertical construction joint the whole of the hoop tension must be taken by the steel and the stress in the steel must be limited, as otherwise it will stretch and allow the joint to open and leak. Likewise the tensile stress in the concrete must also be limited or it will crack and leak. It must be remembered that failure to satisfy all three main considerations stated on page 5 of the previous chapter will result in an unsound structure.

In normal slab-and-beam work the main bars can be extended until their ends lie in a region of little tension and often into a region of compression. No experienced designer would deliberately lap the main tension bars at midspan in a freely-supported beam, but these are exactly the conditions that must be faced in detailing the hoop steel in a circular tank, since the maximum tension occurs at all vertical wall sections around the tank.

There has been much controversy as to the upper limits that should be imposed on the tensile stresses in the steel and concrete. The official view in this country is expressed in the current edition of CP.2007. In the 1960 edition the hoop tension in the steel is limited to 12,000 lb. per sq. in. and the calculated tension on concrete of 1 : 1·6 : 3·2 mix to 190 lb. per sq. in. (presumably related to 200 lb. per sq. in. on a standard 1 : 1·5 : 3 mix). These figures are not accepted by many engineers who have successfully designed and built tanks to higher stresses and it is evident that a hoop tension in the steel of 13,500 lb. per sq. in. and a calculated tension of 300 lb. per sq. in. on 1 : 1·5 : 3 concrete (calculated on the concrete section alone) are quite safe provided that the third principal condition (which is particularly vital in circular tanks) is satisfied, namely the limitation of the bond stress between the steel and the concrete.

The origin of the disagreement on working stresses is probably historical. The wall of at least one of the early circular tanks constructed, subsequently failed and collapsed. This led to panic and a lot of muddled thinking, some of which apparently still persists. If the horizontal steel in the wall of a circular tank is welded to form complete hoops and the concrete is stronger than an average 1 : 3 : 6 mix, the tank wall cannot possibly collapse (although it may crack very badly) until the stress in the steel reaches the yield point (say 40,000 lb. per sq. in. for mild steel). Few, if any, of the early circular tanks were designed for a steel stress in excess of 17,000 lb. per sq. in. with a concrete mix not weaker than 1 : 2 : 4. It is therefore quite clear that any collapse which occurred could not possibly have been caused by overstressing of the steel or the concrete but must have been due to bond failure. Immediate preventative measures taken should not have consisted of a severe reduction in allowable working stresses in the concrete and the reinforcement, but should have consisted of reducing the number of overlaps, distributing them evenly round the tank, increasing the length of overlap provided, and insisting on end anchors on all bars (or alternatively welding all steel bars into complete hoops).

Following the more recent collapse of a prestressed concrete tank there appears to be no suggestion that the working stresses should be reduced and the minimum diameter of the reinforcement increased or that better methods of grouting and protecting the steel should be insisted on.

If the bottom of the wall in *Fig.* 2.1 is designed to the requirements of CP.2007: 1960 (taking D as 50 ft., H as 15 ft. and using a 1 : $1\frac{1}{2}$: 3 mix),

$$\text{maximum hoop tension} = 62{\cdot}5 \times 15 \times 50 \times \tfrac{1}{2}$$

$$= 23{,}400 \text{ lb. per foot height of wall.}$$

Area of hoop steel required $= \frac{23{,}400}{12{,}000} = 1{\cdot}95$ sq. in. per foot height of wall.

If t is the thickness of the wall in inches the tensile stress in the concrete is $\frac{23{,}400}{12\,t + [(15 - 1) \times 1{\cdot}95]}$ which must not exceed 200 lb. per sq. in., giving a minimum value of t of 7·48 in., say an 8-in. wall. Increasing the allowable hoop tension to 13,500 lb. per sq. in. would reduce the area of hoop steel required from 1·95 to 1·74 sq. in. (see *Fig.* 2.2). Checking the condition that the tensile stress on the concrete, neglecting the steel, should not exceed 300 lb. per sq. in.,

$$\text{minimum } t = \frac{23{,}400}{300 \times 12} = 6{\cdot}5 \text{ in., say a 7-in. wall.}$$

Coming to the third vital condition, CP.2007:1960 allows a bond stress of 130 lb. per sq. in. with a 1 : 1·6 : 3·2 mix, irrespective of whether this bond stress is developed in bending or in hoop tension. Taking this figure at its face value and using a value of 135 lb. per sq. in. for 1 : $1\frac{1}{2}$: 3 concrete gives a calculated grip length of only 22·2 diameters, which is less than the specified alternative minimum of 30 diameters, and this, apparently, is considered adequate even if the bars have no end hooks. If a standard U-hook is provided this may be taken as equal to a length of 16 diameters and an overlap of only 14 diameters is then apparently considered safe.

Examination of a reinforced concrete container which had been partially wrecked by an accidental internal explosion showed that the shock had destroyed the bond between the concrete and the straight lengths of bar but that the end hooks had held, saving the structure from total collapse.

The writer would not accept any arrangement of unwelded bars for hoop steel in a circular tank unless:

(*a*) all bars have standard U-hooks at both ends.
(*b*) The overlap (excluding end hooks) is not less than 40 diameters nor less than 2 ft.
(*c*) Not more than one bar in three is lapped at any vertical section.

However, with the foregoing provisos the writer would certainly accept a limiting tensile stress of 13,500 lb. per sq. in. Adding an overlap of 26 diameters to a $\frac{3}{4}$-in. bar 40 ft. long, increases the amount of steel provided by about 4 per cent.: raising the stress from 12,000 to 13,500 lb. per sq. in. saves about 12 per cent. If the labour required to form the end hooks is considered the two costs would be about the same. There can be no doubt as to which is the sounder job.

If the tensile stress on the concrete at the base of the wall is 200 lb. per sq. in., $E_c = 3{,}000{,}000$ lb. per sq. in. and the diameter of the tank is 50 ft. then the base of the wall moves outwards a distance of $\frac{f_{ct}}{E_c} \times \frac{D}{2} = \frac{200}{3{,}000{,}000} \times \frac{50}{2}$ ft. = 0·02 in. only. Since the pressure of the water varies directly as the depth below the surface the thickness of the upper part of the wall could be reduced thus saving concrete at the expense of complicating the shuttering, provided that the wall is not made so thin that water actually seeps through it. Many small early tanks had walls 5 in. thick and some were only 4 in., but the usual minimum thickness is now 6 in. A tank only 50 ft. in diameter and 15 ft. deep is too small to have a tapering or stepped wall but the walls of large tanks are generally tapered or stepped-in at the top (see Chapter 6).

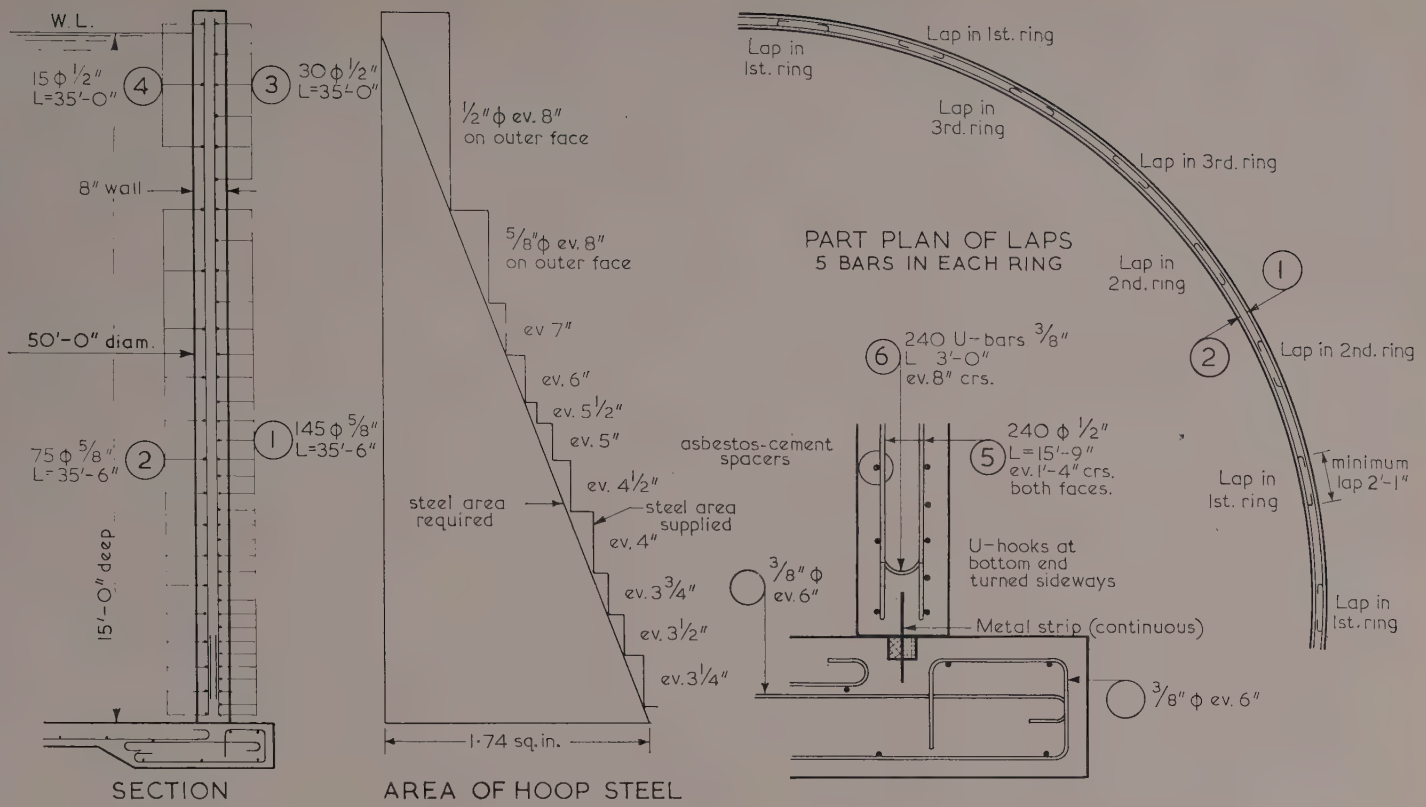

W.L.
15 φ ½" L=35'-0"
4
3
30 φ ½" L=35'-0"
8" wall
50'-0" diam.
75 φ ⅝" L=35'-6"
2
1
145 φ ⅝" L=35'-6"
15'-0" deep
SECTION
½" φ ev. 8" on outer face
⅝" φ ev. 8" on outer face
ev 7"
ev. 6"
ev. 5½"
ev. 5"
ev. 4½"
steel area required
steel area supplied
ev. 4"
ev. 3¾"
ev. 3½"
ev. 3¼"
1·74 sq. in.
AREA OF HOOP STEEL
Lap in 1st. ring
Lap in 1st. ring
Lap in 3rd. ring
Lap in 3rd. ring
PART PLAN OF LAPS
5 BARS IN EACH RING
Lap in 2nd. ring
1
Lap in 2nd. ring
2
6
240 U-bars ⅜" L 3'-0" ev. 8" crs.
asbestos-cement spacers
5
240 φ ½" L=15'-9" ev. 1'-4" crs. both faces.
minimum lap 2'-1"
Lap in 1st. ring
U-hooks at bottom end turned sideways
⅜" φ ev. 6"
Metal strip (continuous)
Lap in 1st. ring
⅜" φ ev. 6"

Fig. 2.2.

Arrangement of Reinforcement

There is a considerable lack of agreement between the methods used by different designers for placing the hoop and vertical reinforcement. In many cases the vertical bars are arranged outside the rings of horizontal steel, whereas the labour in fixing the bars could be reduced by having the rings on the outer side, thus avoiding the need to lift the heavy hoops over the vertical bars or to thread them through.

Despite the recommendations of CP.2007:1960 hooping-bars are generally lapped for a length of 40 diameters and hooked at both ends, even when the steel stress is limited to 12,000 lb. per sq. in. The laps are also tied with 16-gauge soft iron wire to prevent the bars being disturbed during the process of placing the concrete. The practice of winding the tying wire around the whole length of the lap should not be permitted, as this wire prevents access of the concrete to the lapped rods. Some engineers believe that the best position for the hooping steel is near the outside of the wall, particularly if hot liquids are to be stored in the tank, but in large deep tanks it is an advantage to have an inner and an outer ring of steel in order to avoid the need to bend and fix very heavy bars. Many engineers use only a single ring in comparatively small tanks, but a better arrangement near the bottom of the wall is sometimes obtained by inserting two series of hoops and staggering them vertically. When the spacing is close and the bars are in a single ring, the steel is crowded in the vicinity of each lap and, if these are made in the customary fashion, there may be quite an appreciable length of bar not in contact with the concrete, particularly when a dry concrete mix is used. The strength of the lap depends on the adhesion between the concrete and the steel, and if this adhesion is absent from even a small portion of the reinforcing bar the strength of the joint may be reduced to that of, say, a length of 20 bar diameters. In such an event the concrete is liable to crack.

Bars bent to a radius must be properly curved in a hooping machine, and not bent in a series of kinks by an ordinary bar-bending machine. Since no question of eccentricity of pull arises if the bars are properly pulled round or hooped by machine, the best joint would appear to be one in which the bars pass but do not touch one another at their ends and are wired to the verticals in such a way as to allow the concrete to have free access between them.

The failure of a reservoir 40 ft. in diameter in Australia was largely attributed to the use of a single layer of hoop steel and a lack of adhesion at the joints due to the concrete being too harsh. At the present time this failure should be borne in mind when it is proposed to use stiff vibrated concrete. It is noteworthy that a very wet concrete may also cause a loss of adhesion by falling away from below the bars and leaving only water in contact with the steel.

Shortly after the construction of a circular reservoir in the United States some years ago, vertical cracks appeared on the outer surface all round the wall at intervals of 20 ft. to 25 ft. These cracks were most apparent at the bottom where the thickness of the concrete was greatest and were attributed to the fact that, although the steel was arranged in two rings, these were very close to the inner face of the wall. In this case the cracks did not penetrate to the inner face. The writer usually compromises and puts two-thirds of the hoop steel in the outer ring and one-third near the liquid face. A proposed detail for the wall of a tank 50 ft. in diameter and 15 ft. deep is shown in *Fig.* 2.2. The vertical steel is 0·3 per cent. of the gross concrete section. This detail is based on a stress in the hoop steel

of 13,500 lb. per sq. in. The reader may re-design it to comply with the latest version of CP.2007. (Such official documents may change considerably from one edition to the next.) The wall is made 8 in. thick to accommodate the sliding joint detail, which is similar to that shown in *Fig.* 2.8.

Early Tanks

The provision of a sliding joint at the base of the wall is a comparatively recent innovation. Most early circular tanks were designed on the assumption that the wall could be designed as an independent unit unconnected with the floor although no sliding joint was actually provided. In most cases, however, the instinct of the designer led him to stiffen and strengthen the joint between wall and floor. Many early tanks were given splays in the internal corner and some had splays both inside and out as shown in *Fig.* 2.3. These splays were provided with corner

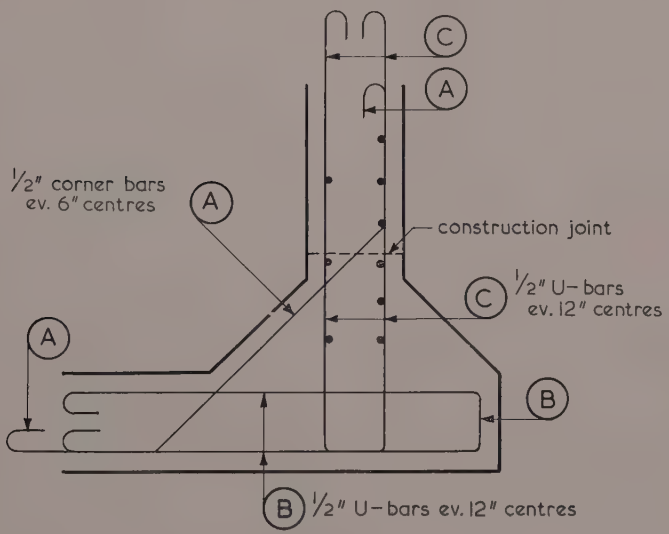


Fig. 2.3.

bars which were put in by eye. One would not reproduce this form of construction today—the tank wall would either have a sliding joint or be designed for a calculated restraint moment (as described in the following chapters). Nevertheless these early tanks served their purpose, as do most structures that are designed more by instinct than by mathematics. Modern tanks only have corner splays if the use to which they are to be put demands them. If provided they should be cast integral with the floor, the construction joint being in the position shown in *Fig.* 2.3.

In *Fig.* 2.4 an actual tar tank is shown in which the joint at the base of the wall has been strengthened by a square thickening. Note that if the bars marked 105 and 106 were provided with end hooks and closed to 6-in. spacing, the wall would satisfy the condition of total restraint (see Chapter 3).

Accidental Overload

Most open-topped tanks have only a few inches of freeboard and even if the normal overflow outlets get choked the amount of accidental overload is small.

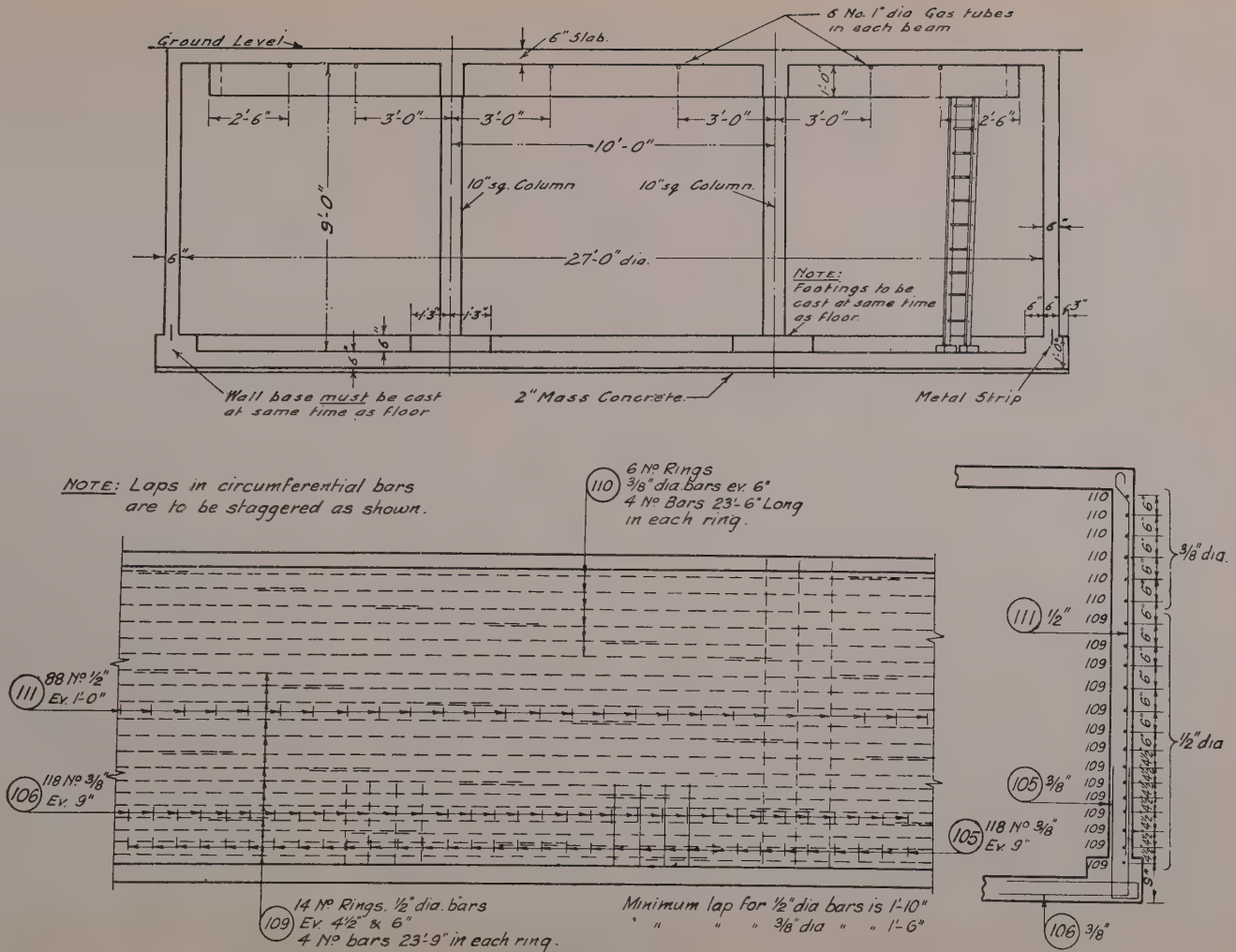

Ground Level
6" Slab.
8 No. 1" dia Gas tubes in each beam
2'-6"
3'-0"
3'-0"
3'-0"
3'-0"
2'-6"
10'-0"
1'-0"
9'-0"
10" sq. Column
10" sq. Column.
6"
6"
27'-0" dia.
NOTE: Footings to be cast at same time as floor.
4 1/2"
4 1/2"
6"
6"
6"
6"
3"
1'-0"
Wall base must be cast at same time as floor
2" Mass Concrete.
Metal Strip
NOTE: Laps in circumferential bars are to be staggered as shown.
110 6 No Rings 3/8" dia Bars ev. 6" 4 No Bars 23'-6" Long in each ring.
111 88 No 1/2" Ev. 1'-0"
106 118 No 3/8" Ev. 9"
109 14 No Rings. 1/2" dia. bars Ev. 4 1/2" & 6" 4 No bars 23'-9" in each ring.
Minimum lap for 1/2" dia bars is 1'-10"
" " " " 3/8" dia " " 1'-6"
110 110 110 110 110 110 109 109 109 109 109 109 109 109 109 109 109 109 109 109 109
3/8" dia.
111 1/2"
1/2" dia
105 3/8"
105 118 No 3/8" Ev. 9"
106 3/8"

Fig. 2.4.

If the freeboard exceeds a few inches the designer should check the calculated hoop tension assuming that the overflows are choked and the water-level rises to the top of the wall. Under these conditions the calculated stress in the hoop steel should not exceed 17,000 lb. per sq. in.

If there is any likelihood that the tank may be used to store a much heavier liquid it must be designed accordingly. It would be most unsafe to put cement slurry weighing 100 lb. per cu. ft. into a tank designed to hold water.

Shrinkage and Temperature Stresses

Immediately the walls are completed a foot or so of water should be put into the bottom of the tank and the wall kept damp for a week at least. If a tank has been left standing empty in a hot sun for several months it should be filled very slowly and the wall sprayed with water since concrete, even when well matured, may shrink when thoroughly dried out and expand again when thoroughly soaked. Shrinkage induces compression in the steel and tension in the concrete. When a dried-out tank is filled quickly all the tension may come on to the concrete before the steel can start to function; hence the criterion that the tensile stress in the concrete alone, ignoring the steel, should not exceed 300 lb. per sq. in.

When the wall has a sliding joint at the bottom the wall merely moves out or in as it expands or contracts with temperature or shrinkage changes and no appreciable stresses are produced (assuming that the joint functions as intended).

If the temperature of the wall rises by 100 deg. F. and the coefficient of linear expansion is 0·000006 per deg F., the radius of 25 ft. in *Fig.* 2.2 will increase by 100 × 0·000006 × 25 ft., or 0·18 in. Assuming that the floor does not expand, this movement is nine times as great as the outward movement previously calculated as being caused by the water pressure. If the tank has to hold warm liquid the sliding joint must accommodate this movement and the materials used in the joint must not soften unduly with the heat.

Sliding Joints

As the wall moves outwards some friction must occur between the bottom edge of the wall and the top face of the floor slab, and this frictional force tends to tear

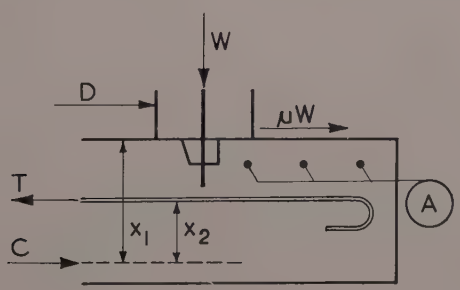


Fig. 2.5.

away the edge of the floor; the action is as shown in *Fig.* 2.5. If W is the weight of the wall per foot run and μ is the coefficient of friction, then the bottom edge of the wall applies an outward drag of μW per foot run as the tank is filled. (To

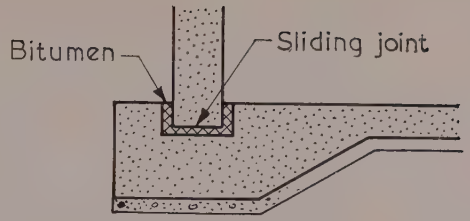


Fig. 2.6

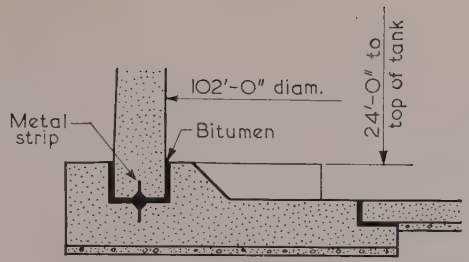
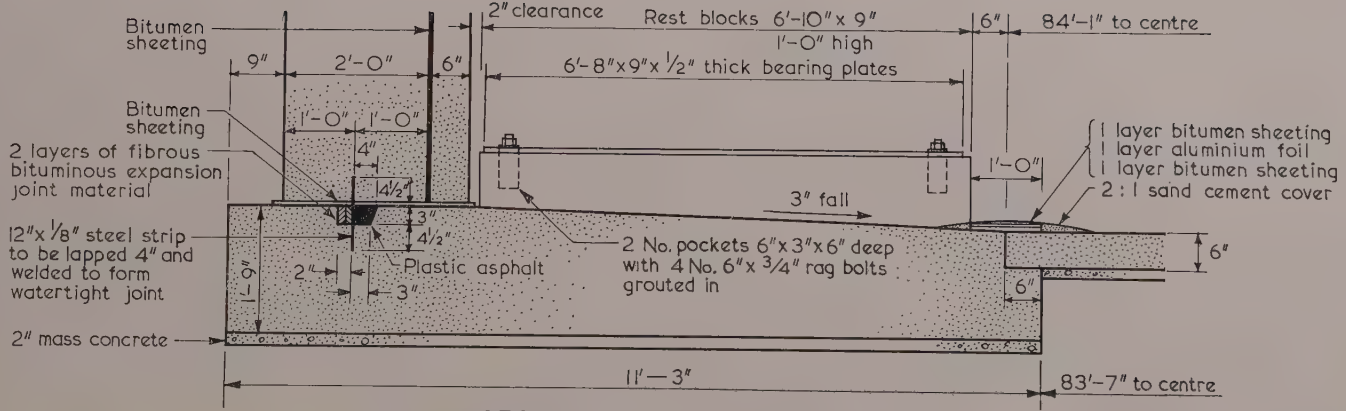


Fig. 2.7.



SECTION THROUGH RING BEAM

Fig. 2.8.

this should be added the resistances of the water-bar plate and the joint filling, but these are probably small.) The outward drag of μW can be resisted by the forces T and C shown in *Fig.* 2.5, where C is the compression in the concrete. Taking moments about the line of action of C,

$$T = \mu W x_1 / x_2,$$

and if f_{st} is the allowable tensile stress in the steel and A_{st} is the area of steel required per foot run of wall,

$$A_{st} = \mu W x_1 / f_s\, x_2.$$

If $\mu = 1{\cdot}0$, $W = 1{,}550$ lb. per ft. run, $x_1 = 10$ in., $x_2 = 5$ in. and $f_{st} = 18{,}000$ lb. per sq. in.,

$$A_{st} = \frac{1{\cdot}0 \times 1{,}550 \times 10}{18{,}000 \times 5} = 0{\cdot}172 \text{ sq. in. per foot run of wall.}$$

Alternatively rings of steel (marked A in *Fig.* 2.5) can be supplied to resist the force μW. If D is the diameter of the tank, the total tension in these bars is very nearly $\frac{1}{2}\mu WD$. If $D = 50$ ft. and taking the values already assumed:

$$\text{Area of hoop steel required} = \frac{1{\cdot}0 \times 1{,}550 \times 50}{18{,}000 \times 2} = 2{\cdot}15 \text{ sq. in.},$$

or, say 3 rings of 1-in. diameter bars.

Synthetic products are now obtainable which have small values of μ. Values of μ of less than 0·2 are claimed for some products. A sheet of such material may be used on one side of the joint with a sheet of stainless steel on the other to give a material-to-steel sliding contact. The use of multi-layer rubber pads has been suggested but the extent of their effective life seems doubtful.

Early types of sliding joint are shown in *Figs.* 2.6 and 2.7. The jointing material is described as 'bitumen'. The designer apparently hoped that the 'bitumen' would be sufficiently stiff to carry the weight of the wall without squeezing out, but would be soft enough to squeeze out at once immediately the tank wall started to move laterally. A more recent detail is shown in *Fig.* 2.8.

If the material in the sliding joint breaks down altogether and the joint does not slide at all, the case of a restrained wall hinged at the base may be reached. This case is discussed in Chapter 3. Taking an approximate value for the moment in the wall of $0{\cdot}10wHTR$ for the tank in *Fig.* 2.2,

$$M = 0{\cdot}10 \times 62{\cdot}5 \times 15 \times 0{\cdot}667 \times 25 = 1{,}560 \text{ lb.-ft. per ft.} = 18{,}700 \text{ lb.-in. per ft.}$$

$$\text{Vertical steel required} = \frac{18{,}700}{12{,}000 \times 0{\cdot}9 \times 5{\cdot}5} = 0{\cdot}315 \text{ sq. in.}$$

The detail in *Fig.* 2.2 shows $\frac{1}{2}$-in. vertical bars at 16-in. centres near both faces. If intermediate $\frac{1}{2}$-in. bars at 16-in. centres are added there are $\frac{1}{2}$-in. bars at 8-in. centres or 0·29 sq. in. per foot. These intermediate bars need run only halfway up the wall and would only be required if the joint should break down completely.

In all cases of sliding, rolling or hinged joints it is prudent to calculate what would happen if the joint seized up and failed to slide, roll or rotate. To what extent and at what stresses this contingency should be allowed for must depend on site conditions and on the relative importance of the structure. For the tank in *Fig.* 2.2, if the contingency may be covered by permitting an overstress of 20,000 lb. per sq. in. instead of the usual stress of 12,000 lb. per sq. in., the area of steel required is reduced from 0·315 to 0·19 sq. in. If the number of vertical

bars marked 5 were increased from 240 to 320, their spacing would be 12 in. instead of 16 in. and their area 0·196 sq. in. per foot run of wall. Whether this could be regarded as a reasonable precaution against failure of the sliding joint would depend on site conditions and especially on the exact type of material used to form the joint.

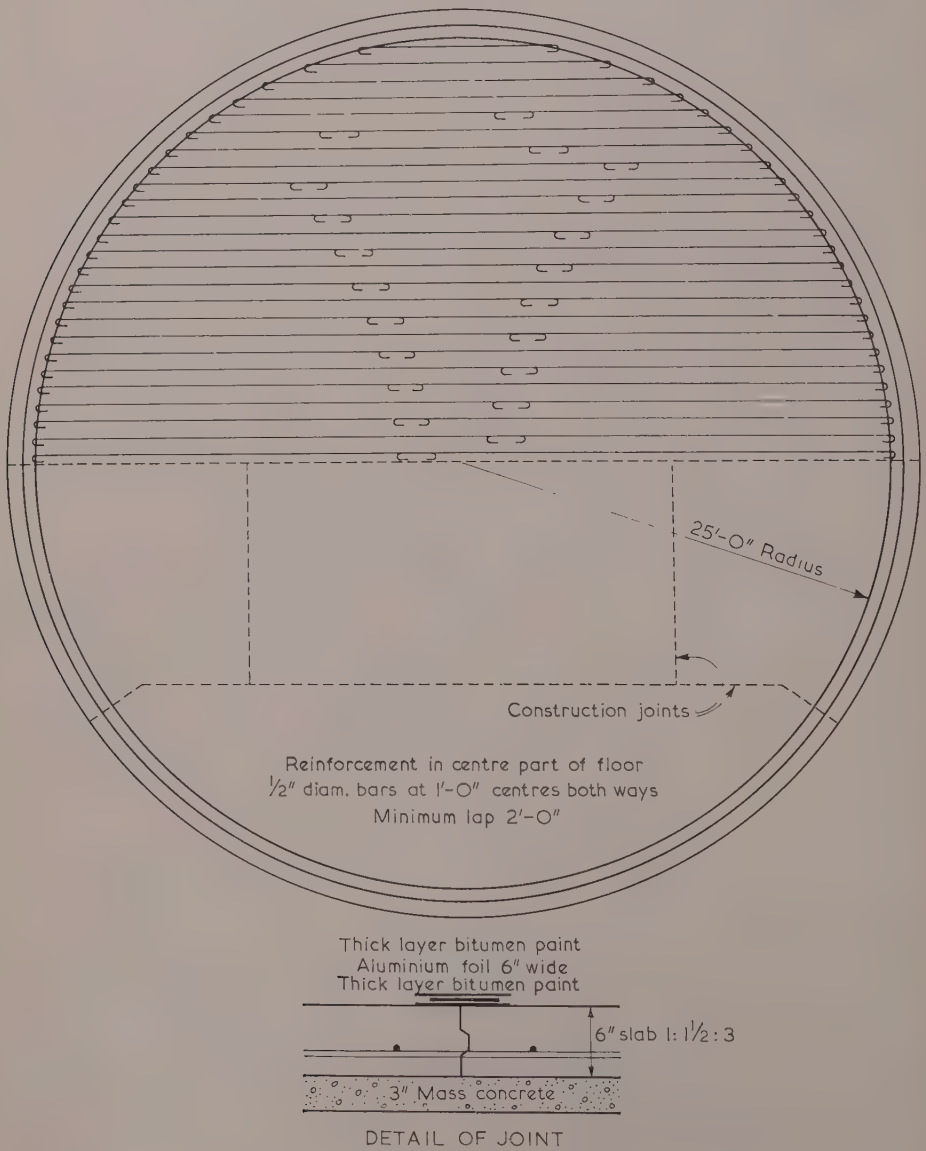

25'-0" Radius
Construction joints
Reinforcement in centre part of floor
½" diam. bars at 1'-0" centres both ways
Minimum lap 2'-0"
Thick layer bitumen paint
Aluminium foil 6" wide
Thick layer bitumen paint
6" slab 1: 1½: 3
3" Mass concrete
DETAIL OF JOINT

Fig. 2.9.

Tank Floors

The general design of tank floors is discussed in Chapter 16. Assuming the tank in *Fig.* 2.2 stands on firm dry ground, the steel in the central part of the floor may be about 0·3 per cent. of the concrete section, as shown in *Fig.* 2.9. Beginners, when detailing circular work, are apt to have a multiplicity of bar lengths varying by increments of 1 in. Apart from the additional labour that would be needed on the site to sort out the bars, such an idea will not work as the lengths of the bars are often incorrect to a tolerance of ±1 in. and a bar which has been ordered 24 ft. 11 in. long may actually be 25 ft. 0 in. in length while a bar which should be 25 ft. 0 in. long may also be exactly 25 ft. 0 in. in length. Many designers work to increments of not less than 3 in. and some to increments of not less than 6 in.

Practical designs for tank floors are never as simple as that shown in *Fig.* 2.9 as they are laid to fall to an outlet, drainage trench or sump. The use of synthetic water-bars should be avoided in small thin floors.

Economical Dimensions

The dimensions of practical tanks are generally fixed by site conditions—space available, ground level, intake level, outlet and drain levels, etc. Only on rare occasions can the diameter and depth be varied. The tank in *Fig.* 2.2 contains

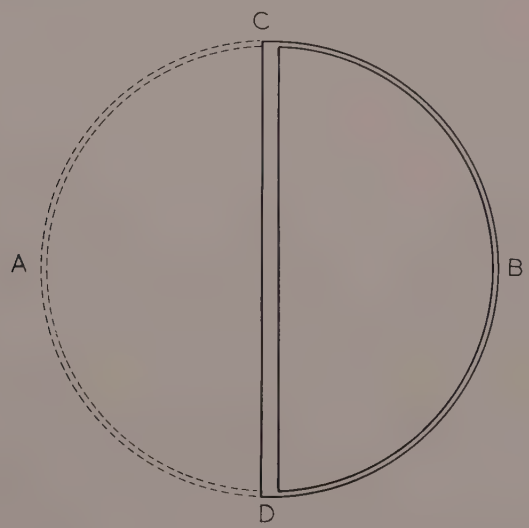


Fig. 2.10.

29,500 cu. ft. The reader may re-design it as 12 ft. deep by 55 ft. 11 in. diameter, 18 ft. deep by 45 ft. 7 in. diameter and 21 ft. deep by 42 ft. 3 in. diameter, and compare the quantities of concrete, steel and shuttering involved in each of the four alternative designs. In practice each price would have to include the cost of excavation, drains, pipework, pumps, etc.

Semicircular Tanks

It is possible to build the semicircular tank CBD in *Fig.* 2.10 provided the wall CD is strong enough to span from C to D, the wall CBD remaining in pure hoop tension.

It is not possible to divide a circular tank into two compartments by building across the tank a straight wall that cantilevers from the floor. If a circular tank must be divided into two compartments a circular wall should be provided inside the tank. This wall is then in pure tension or pure compression depending on which compartment is full and which empty.

CHAPTER THREE

RESTRAINT IN OPEN-TOPPED CIRCULAR TANKS WITH WALLS OF UNIFORM THICKNESS

THE following consists of a collection and amplification of results already published elsewhere.

Assumptions

It is assumed that the upper end of the wall is unrestrained in position and direction and that the wall is of uniform thickness from top to bottom. The usual assumption, that the wall consists of some uniform elastic material, is also made although the designer will, of course, vary the amount of reinforcement from section to section to cover the calculated moments and ring tensions.

One value which appears in the calculations is E, the modulus of elasticity, but this does not occur in most of the results as it cancels out. In this respect it is assumed that the value of E (in the expression EI) used to calculate the moments, shears and deflections is the same as the value of E used to calculate the extension due to hoop tension. With such a variable and unpredictable material as concrete in bulk this assumption is open to question but it is probably more accurate than the assumption of absolute fixity at the base of the wall, where there is usually a major construction joint and a time interval occurs between the placing of adjoining areas of concrete.

The pressure comes on the inside face of the wall and the total water pressure depends on the internal radius R, but when calculating the bending and stretching of the wall really the radius of the centre-line, which is $(R + 0{\cdot}5T)$, should be taken. The error due to this discrepancy is probably very small compared with errors in the primary assumptions made.

The usual tacit assumption is made that Poisson's ratio for reinforced concrete may be taken as zero.

Fundamental Equations

When the wall of a circular tank (*Fig.* 3.1) is attached rigidly to the floor and the floor is held rigidly in position by the ground on which the tank is supported, a vertical section through the wall, when the tank is full, takes the form shown diagrammatically by the curve ADB in *Fig.* 3.1.

If the wall is now separated from the floor by introducing a frictionless sliding joint at the base of the wall, it will take up the shape AC, where the distance BC is equal to $w\frac{HR^2}{ET}$. This distance is very small and it is necessary greatly to exaggerate the horizontal scale before it becomes visible in a diagram.

If w is the weight of fluid retained (=62·5 lb. per cu. ft.), H is the depth in feet (=25 ft.), R is the radius in feet (=30 ft.), T is the wall thickness in feet (=1·17 ft.) and E is the elastic modulus (=2,000,000 × 144 lb. per sq. ft.), then

$$\frac{wHR^2}{ET} = \frac{62{\cdot}5 \times 25 \times 30^2}{2{,}000{,}000 \times 144 \times 1{\cdot}17} = 0{\cdot}0042 \text{ ft.} = 0{\cdot}05 \text{ in.}$$

It is always useful to draw this line AC and compare it with the curve ADB, for the area of triangle ABC represents, to some scale, the total water pressure on the

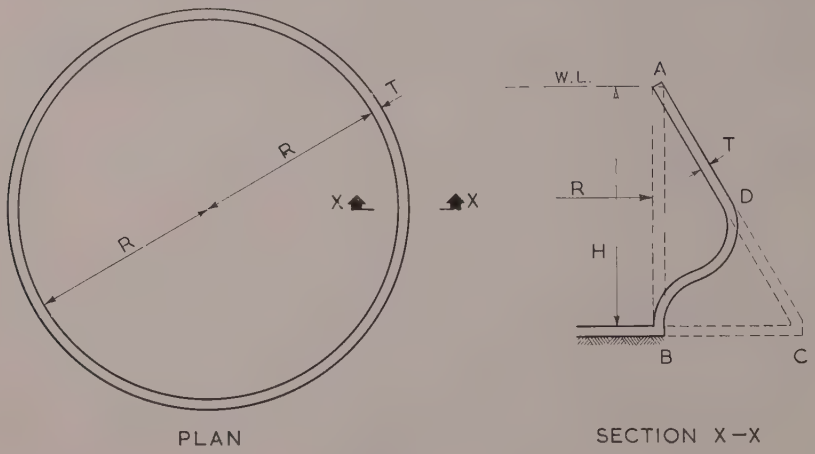


Fig. 3.1.

wall. The area ADB represents the amount of pressure resisted by hoop tension and the area DBC is approximately the amount taken by cantilever action at the base of the wall.

Drawing the wall of the tank as a single line in *Fig.* 3.2 and taking the origin for h (the depth below water level) at the water surface and measuring y (the outward deflection of the wall) from the vertical line AB, the increase in the radius

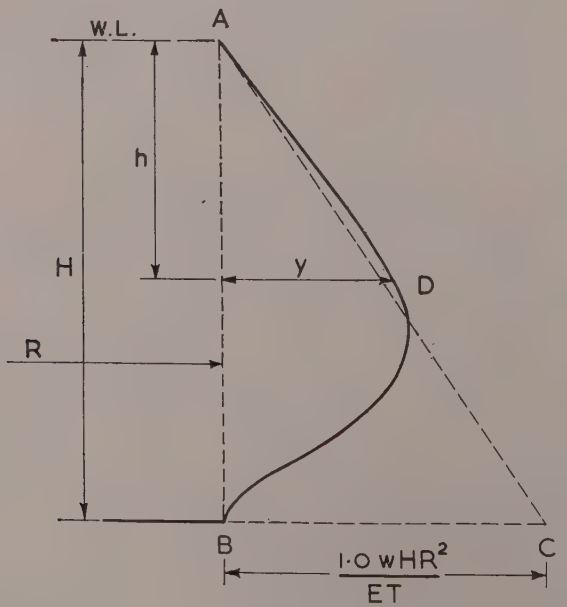


Fig. 3.2.

R is equal to y and the strain at depth h is y/R and the unit hoop tension in the wall at this depth is yE/R. The total hoop tension per unit height of wall is yET/R, where T is the thickness of the wall. This hoop tension could be caused by and corresponds to a water pressure of yET/R^2. The actual applied water pressure at any depth h is wh and the unbalanced horizontal loading is $\left(wh - y\frac{ET}{R^2}\right)$, this unbalanced load being carried by the wall acting as a series of vertical cantilevers fixed at their lower ends into the floor of the tank. The maximum moment and maximum shear on each of these vertical cantilevers occurs at the bottom of the wall and these two values at least must be known before the design of the wall can be commenced.

If the wall of the tank is imagined to be cut into a large number of narrow vertical staves, each acting as a vertical cantilever, the effective loading on these per unit length of perimeter is $\left(wh - y\frac{ET}{R^2}\right)$ and if I is the geometrical moment of inertia of the cross-section of one of these staves per unit length of perimeter:

$$EI\frac{d^4y}{dx^4} = wh - y\frac{ET}{R^2} \text{ or } \frac{d^4y}{dh^4} = \frac{wh}{EI} - \frac{yT}{R^2I}.$$

Solving this gives:

$$y = \frac{wh}{\alpha^4 EI} + \left[e^{\frac{\alpha h}{\sqrt{2}}}\left(A \sin\frac{\alpha h}{\sqrt{2}} + B\cos\frac{\alpha h}{\sqrt{2}}\right) + e^{-\frac{\alpha h}{\sqrt{2}}}\left(C\sin\frac{\alpha h}{\sqrt{2}} + D\cos\frac{\alpha h}{\sqrt{2}}\right)\right]. \quad (1)$$

where $$\alpha = \sqrt[4]{\frac{T}{R^2I}},$$

and the coefficients A, B, C and D are found from the end conditions at the top and bottom of the wall. As a fully-restrained open-topped tank is being considered,

$$\frac{d^2y}{dh^2} = 0 \quad \text{when } h = 0 \quad . \quad . \quad . \quad . \quad (2)$$

$$\frac{d^3y}{dh^3} = 0 \quad \text{when } h = 0 \quad . \quad . \quad . \quad . \quad (3)$$

$$y = 0 \quad \text{when } h = H \quad . \quad . \quad . \quad . \quad (4)$$

$$\frac{dy}{dh} = 0 \quad \text{when } h = H \quad . \quad . \quad . \quad . \quad (5)$$

From equation (2), $C = A$ . . . . . (6)

From equation (3), $D = B - 2A$ . . . . . (7)

This leaves only two coefficients to be calculated and these are found from equations (4) and (5).

Also I per unit length of perimeter $= \frac{1}{12}T^3$, so that

$$\alpha = \sqrt[4]{\frac{T \times 12}{R^2T^3}} = \frac{1{\cdot}86}{\sqrt{TR}}.$$

Differentiating equation (1),

$$\frac{dy}{dh} = \frac{w}{\alpha^4 EI} + \frac{\alpha}{\sqrt{2}}\left[e^{\frac{\alpha h}{\sqrt{2}}}\left(A\sin\frac{\alpha h}{\sqrt{2}} + B\cos\frac{\alpha h}{\sqrt{2}} + A\cos\frac{\alpha h}{\sqrt{2}} - B\sin\frac{\alpha h}{\sqrt{2}}\right)\right.$$
$$\left. + e^{-\frac{\alpha h}{\sqrt{2}}}\left(-C\sin\frac{\alpha h}{\sqrt{2}} - D\cos\frac{\alpha h}{\sqrt{2}} + C\cos\frac{\alpha h}{\sqrt{2}} - D\sin\frac{\alpha h}{\sqrt{2}}\right)\right] \quad . \quad (8)$$

$$\frac{d^2y}{dh^2} = \alpha^2 \left[e^{\frac{\alpha h}{\sqrt{2}}} \left(A \cos \frac{\alpha h}{\sqrt{2}} - B \sin \frac{\alpha h}{\sqrt{2}} \right) + e^{-\frac{\alpha h}{\sqrt{2}}} \left(- C \cos \frac{\alpha h}{\sqrt{2}} + D \sin \frac{\alpha h}{\sqrt{2}} \right) \right] \quad . \quad (9)$$

$$\frac{d^3y}{dh^3} = \frac{\alpha^3}{\sqrt{2}} \left[e^{\frac{\alpha h}{\sqrt{2}}} \left(A \cos \frac{\alpha h}{\sqrt{2}} - B \sin \frac{\alpha h}{\sqrt{2}} - A \sin \frac{\alpha h}{\sqrt{2}} - B \cos \frac{\alpha h}{\sqrt{2}} \right) + e^{-\frac{\alpha h}{\sqrt{2}}} \left(C \cos \frac{\alpha h}{\sqrt{2}} - D \sin \frac{\alpha h}{\sqrt{2}} + C \sin \frac{\alpha h}{\sqrt{2}} + D \cos \frac{\alpha h}{\sqrt{2}} \right) \right] \quad . \quad (10)$$

$$\frac{d^4y}{dh^4} = - \alpha^4 \left[e^{\frac{\alpha h}{\sqrt{2}}} \left(A \sin \frac{\alpha h}{\sqrt{2}} + B \cos \frac{\alpha h}{\sqrt{2}} \right) + e^{-\frac{\alpha h}{\sqrt{2}}} \left[C \sin \frac{\alpha h}{\sqrt{2}} + D \cos \frac{\alpha h}{\sqrt{2}} \right) \right] \quad . \quad (11)$$

Once the values of A, B, C and D are obtained, the moment at any point is equal to $EI\frac{d^2y}{dh^2}$ and the shear at any point is equal to $EI\frac{d^3y}{dh^3}$.

A study of the foregoing equations shows that all tanks which have the same value of the ratio $H/\sqrt{TR}$ have the same coefficients of moment and shear.

EXAMPLE— Assume $H/\sqrt{TR} = 3$. Then $H = 3\sqrt{TR}$. Since $\alpha = 1{\cdot}86/\sqrt{TR}$,

$$\frac{\alpha H}{\sqrt{2}} = \frac{1{\cdot}86}{\sqrt{TR}} \times \frac{3\sqrt{TR}}{\sqrt{2}} = 3{\cdot}95 \text{ radians.}$$

$$e^{\frac{\alpha H}{\sqrt{2}}} = e^{3{\cdot}95} = 51{\cdot}935. \qquad e^{-\frac{\alpha H}{\sqrt{2}}} = e^{-3{\cdot}95} = 0{\cdot}01926.$$

$$\text{Sin } \alpha H/\sqrt{2} = \sin (3{\cdot}95 \text{ rad.}) = - 0{\cdot}7232, \text{ and}$$
$$\cos \alpha H/\sqrt{2} = \cos (3{\cdot}95 \text{ rad.}) = - 0{\cdot}6907.$$

From equations (4), (6) and (7),

$$- \frac{wH}{\alpha^4 EI} = [- 37{\cdot}537A - 35{\cdot}913B] \quad . \quad . \quad . \quad (12)$$

From equations (5), (6) and (7),

$$- 0{\cdot}253 \frac{wH}{\alpha^4 EI} = [- 73{\cdot}50A + 1{\cdot}677B] \quad . \quad . \quad . \quad (13)$$

From equations (12) and (13),

$$A = + 0{\cdot}00398 \frac{wH}{\alpha^4 EI} \text{ and } B = + 0{\cdot}02372 \frac{wH}{\alpha^4 EI}.$$

From equations (6) and (7),

$$C = + 0{\cdot}00398 \frac{wH}{\alpha^4 EI} \text{ and } D = + 0{\cdot}0158 \frac{wH}{\alpha^4 EI}.$$

Using equation (9) and writing $h = H$,

$$\text{moment at base of wall} = EI\frac{d^2y}{dh^2} = EI\alpha^2\left[+0{\cdot}7483\frac{wH}{\alpha^4 EI}\right] = +0{\cdot}7483\frac{wH}{\alpha^2}.$$

Since $\alpha = 1{\cdot}86/\sqrt{TR}$, this moment is equal to $0{\cdot}216wHTR$.

Using equation (10) and writing $h = H$,

$$\text{shear at base of wall} = EI\frac{d^3y}{dh^3} = \frac{EI\alpha^3}{\sqrt{2}}\left[1{\cdot}75\frac{wH}{\alpha^4 EI}\right] = 0{\cdot}665wH\sqrt{TR}.$$

Since, in this example $H/\sqrt{TR} = 3$, the shear at the base of the wall is equal to $0{\cdot}222wH^2$, that is 44 per cent. of the shear at the base of a cantilever wall.

If the moment and shear at the base of the wall are known the moments and hoop tensions in other parts of the wall can approximately be estimated. This may be sufficient for a small tank but, for a large tank, more exact values may be required. Once the values of A, B, C and D have been found, the deflected shape of the wall can be obtained [from equation (1)], the moment [from equation (9)], and the shear [from equation (10)] at all points along the wall by writing $\alpha h/\sqrt{2} = 0$, 0·2, 0·4, 0·6, etc., up to $\alpha h/\sqrt{2} = 3{\cdot}95$, which occurs when $h = H$.

This rather lengthy process can be much shortened by tabulating the values. For the example being considered, the deflected shape of the wall is as shown in *Fig.* 3.5 and the moments in the vertical staves are shown in *Fig.* 3.9. Since stress and strain are proportional the deflected shape of the wall represents, to some scale, the hoop tension at all levels.

The outward movement of the top of the wall is obtained by putting $h = 0$ in equation (1).

$$y = B + D, \text{ and as } D = B - 2A,\ y = 2(B - A).$$

For this particular value of $H/\sqrt{TR} = 3$,

$$y \text{ at top of wall} = 2(0{\cdot}02372 - 0{\cdot}00398)\ \frac{wH}{\alpha^4 EI} = 0{\cdot}03948\ \frac{wHR^2}{ET}.$$

Shape of Wall

There can scarcely be said to be any ' typical ' shape for a restrained circular wall. In *Fig.* 3.3 are shown the shapes of restrained walls with ratios $H/\sqrt{TR} = 1{\cdot}0$ and $H/\sqrt{TR} = 1{\cdot}5$. For comparison this figure also shows the outward movement of a free cantilevered wall having the ratio $H/\sqrt{TR} = 1{\cdot}0$. The outward deflection is shown in terms of wHR^2/ET. This is the extension that takes place at the base of a wall with a sliding joint and corresponds to a hoop tension of wHR/T. If the outward movement of the wall at any level is $0{\cdot}5wHR^2/ET$, the hoop tension at that level is $0{\cdot}5wHR/T$.

For both of the walls shown in *Fig.* 3.3, the maximum outward movement occurs at the top of the wall and the maximum hoop tension occurs where the water pressure is zero. For the ratio $H/\sqrt{TR} = 1{\cdot}0$, the outward movement of the top of the wall is $0{\cdot}196wHR^2/ET$ and the hoop tension at the top of the wall is therefore $0{\cdot}196wHR/T$. The outward deflection for a free cantilever having the ratio $H/\sqrt{TR} = 1$ is also shown in *Fig.* 3.3 by means of the broken line. For a cantilever

carrying a triangular load the deflection is $\Delta = WH^3/15EI$. Considering a cantilever of unit width and writing $I = \frac{1}{12}T^3$,

$$\Delta = \tfrac{1}{15} \times \tfrac{1}{2} \times wH^2 \times \frac{H^3 \times 12}{ET^3} = 0{\cdot}40\frac{wH^5}{ET^3}.$$

Since $H/\sqrt{TR} = 1$, $H^4 = T^2R^2$ and $\Delta = 0{\cdot}40wHR^2/ET$. In other words if a circular wall having the ratio $H/\sqrt{TR} = 1{\cdot}0$ is taken, all the hoop steel is removed and the wall is cut into a series of narrow vertical cantilevers, the outward movement of the top of the wall would then increase from $0{\cdot}196wHR^2/ET$ to $0{\cdot}40wHR^2/ET$, or to about twice as much.

With a ratio of $H/\sqrt{TR}$ of 1·5, the outward movement of the top of the wall is $0{\cdot}286wHR^2/ET$. If the hoop steel is removed from this wall and it is cut

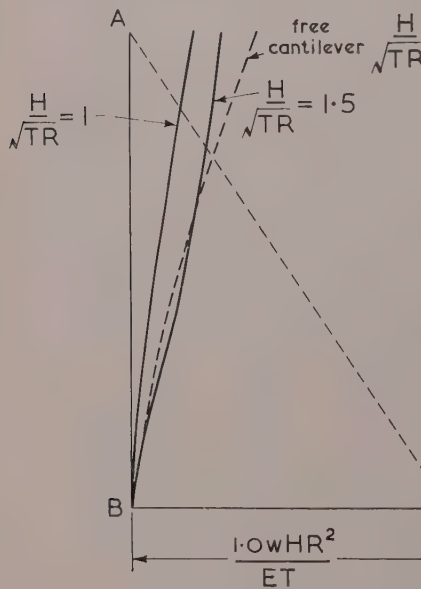


Fig. 3.3.

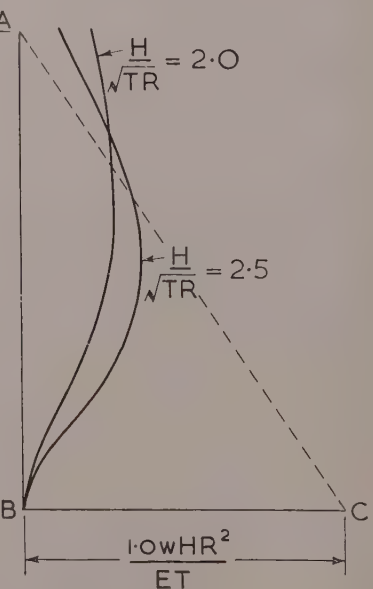


Fig. 3.4.

into narrow vertical cantilevers, the deflection at the top would then increase to $2{\cdot}02wHR^2/ET$ or seven times as much.

In *Fig.* 3.4, the deflected shapes of walls with ratios $H/\sqrt{TR} = 2{\cdot}0$ and $H/\sqrt{TR} = 2{\cdot}5$ are shown. The centre of the wall is now beginning to belly out and the maximum outward movement no longer occurs at the top of the wall.

In *Fig.* 3.5, the corresponding shapes of walls with ratios $H/\sqrt{TR} = 3{\cdot}0$ and $H/\sqrt{TR} = 4{\cdot}0$ are shown. With $H/\sqrt{TR} = 4{\cdot}0$, the top of the wall has returned to its original unloaded position at A. (It has, in fact, passed slightly inside A.)

The deflected shapes of walls with ratios $H/\sqrt{TR} = 6$ and $H/\sqrt{TR} = 8$ are shown in *Fig.* 3.6. The top half of the wall is now in the same state as the top half of a wall with a sliding joint at the base. With $H/\sqrt{TR} = 8$, the maximum outward deflection is 80 per cent. of the distance BC and this maximum deflection occurs close to the floor of the tank.

Although there is resemblance between *Figs.* 3.3 and 3.4, between *Figs.* 3.4 and 3.5, and between *Figs.* 3.5 and 3.6, there is no direct resemblance between *Figs.* 3.3 and 3.6.

In *Fig.* 3.7 the bending moments in the vertical 'staves' of tanks with ratios of $H/\sqrt{TR} = 1{\cdot}0$ and $H/\sqrt{TR} = 1{\cdot}5$ are shown. The broken line on the figure shows the moment in a free cantilevered wall with the ratio $H/\sqrt{TR} = 1{\cdot}0$. The moment at the base of a cantilever of unit width is $0{\cdot}167wH^3$. If $H/\sqrt{TR} = 1{\cdot}0$, then $H^2 = TR$ and $0{\cdot}167wH^3 = 0{\cdot}167wHTR$. The moment at the bottom of a circular restrained wall with $H/\sqrt{TR} = 1$ is $0{\cdot}1065wHTR$ or 64 per cent. of the moment in a free cantilevered wall. For a free cantilevered wall with $H/\sqrt{TR} = 1{\cdot}5$, the moment is $0{\cdot}376wHTR$. The moment at the base of a circular restrained wall

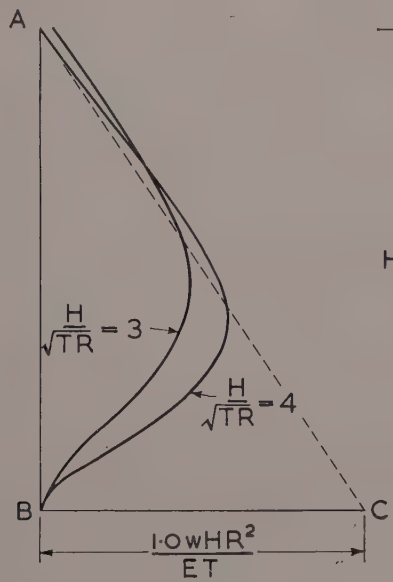


Fig. 3.5.

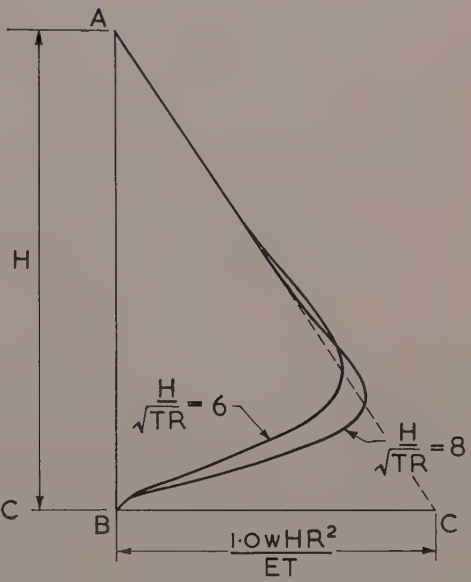


Fig. 3.6.

with $H/\sqrt{TR} = 1{\cdot}5$ is $0{\cdot}1393wHTR$ or 37 per cent. of the moment in a free cantilever with the same ratio.

With a ratio of $H/\sqrt{TR} = 1{\cdot}5$ there is already an appreciable reversed moment (tension on the outside face) in the upper part of the wall.

The moments in tanks with ratios of $H/\sqrt{TR} = 2{\cdot}0$ and $H/\sqrt{TR} = 2{\cdot}5$ are shown in *Fig.* 3.8. Compared with *Fig.* 3.7 the moments are increasing and the position of the maximum reversed moment is falling. In *Fig.* 3.9, which deals with cases where $H/\sqrt{TR} = 3$ and $H/\sqrt{TR} = 4$, the tendencies shown in *Fig.* 3.8 are continued. Moments for ratios of $H/\sqrt{TR} = 6$ and $H/\sqrt{TR} = 8$, are shown in *Fig.* 3.10. As might be deduced from *Fig.* 3.6 there is no appreciable moment in the top half of the wall for these values.

The shear force in the wall is generally only of importance at the base of the wall, where it imposes a considerable tension on the floor of the tank. In *Fig.* 3.2 the area ADB represents, to some scale, the amount of water pressure resisted by hoop tension, leaving the area DBC, approximately, to be taken in shear at point B.

In *Fig.* 3.6 the area DBC for $H/\sqrt{TR} = 8$ is about

$$\tfrac{1}{2} \times 1{\cdot}0\frac{wHR^2}{ET} \times 0{\cdot}2H = 0{\cdot}1\frac{wH^2R^2}{ET}.$$

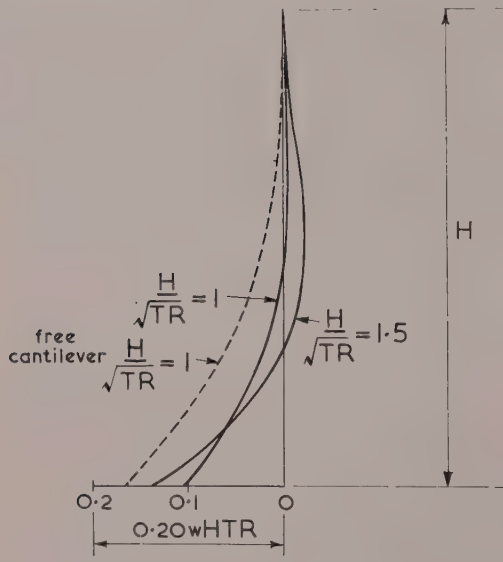


Fig. 3.7.

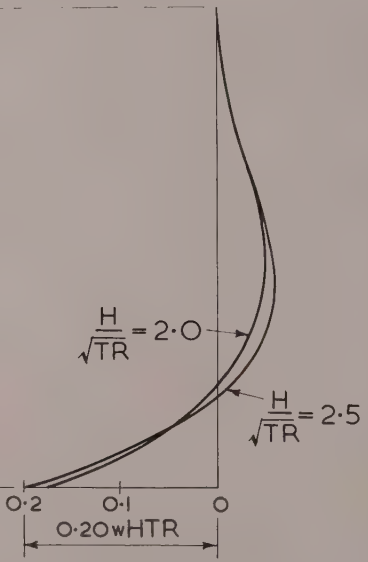


Fig. 3.8.

The total area ABC is $\tfrac{1}{2} \times H \times 1{\cdot}0\frac{wHR^2}{ET} = 0{\cdot}5\frac{wH^2R^2}{ET}$ and this represents the whole water pressure $0{\cdot}5wH^2$, so $0{\cdot}1wH^2R^2/ET$ represents $0{\cdot}1wH^2$. The actual calculated value of the shear for $H/\sqrt{TR} = 8$ is $0{\cdot}0905wH^2$. A complete range

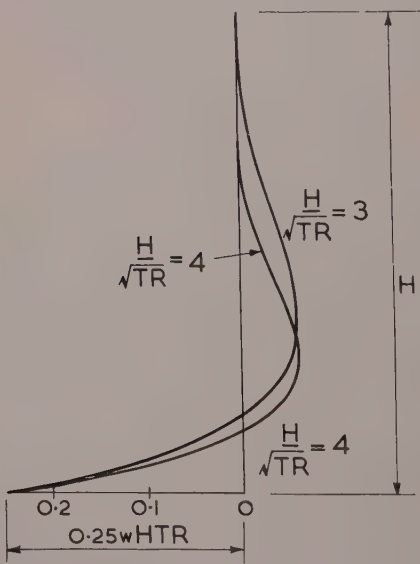


Fig. 3.9.

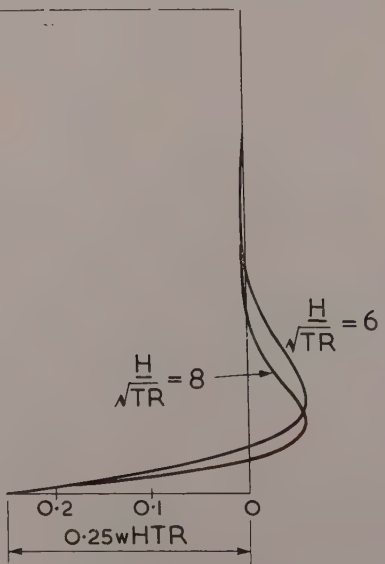


Fig. 3.10.

of values for the shear at the base of the wall is shown in *Fig.* 3.11. In *Figs.* 3.12 and 3.13 the moment at the base of the wall is shown expressed in different terms. For small values of $H/\sqrt{TR}$, *Fig.* 3.12 is more useful, while *Fig.* 3.13 is more useful

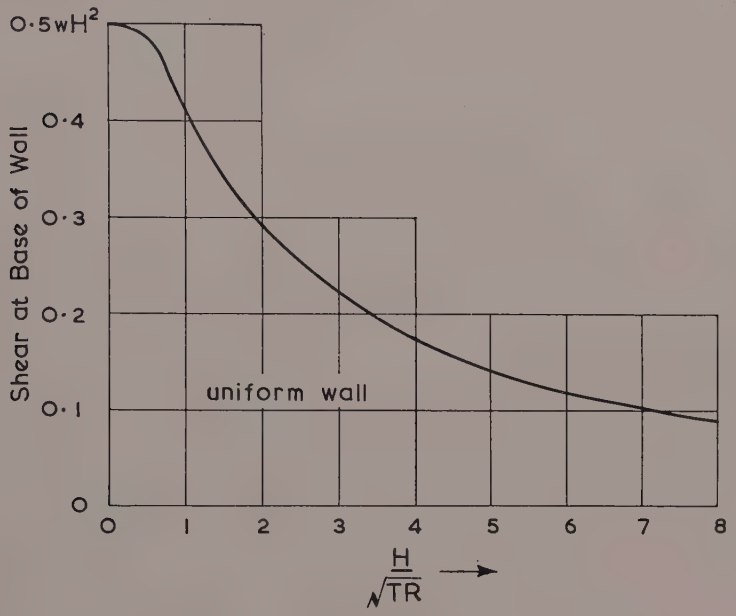


Fig. 3.11.

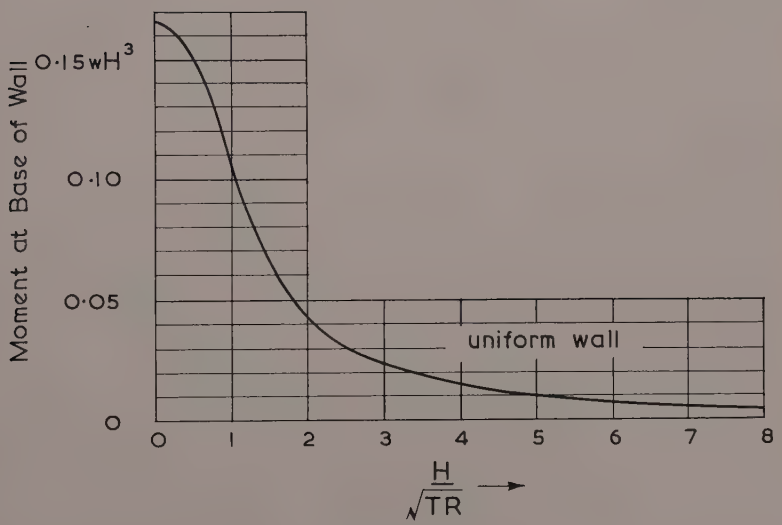


Fig. 3.12.

for larger values as it is easier to read. The maximum value of the reversed moment is given in *Fig.* 3.14 and the position where this maximum moment occurs is shown in *Fig.* 3.15.

The maximum hoop tension is shown in *Fig.* 3.16. The curious shape of this

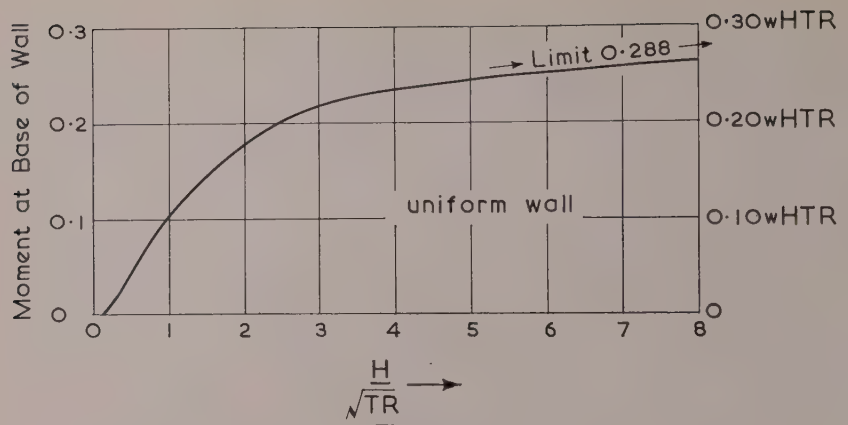


Fig. 3.13.

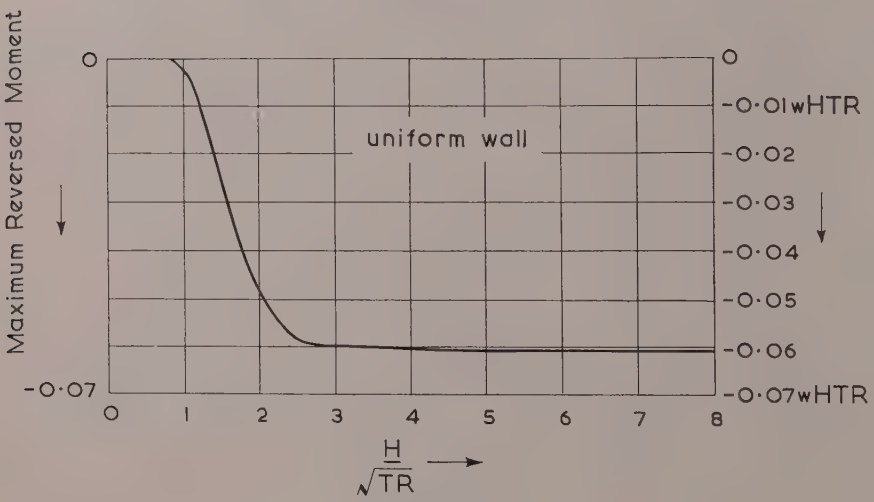


Fig. 3.14.

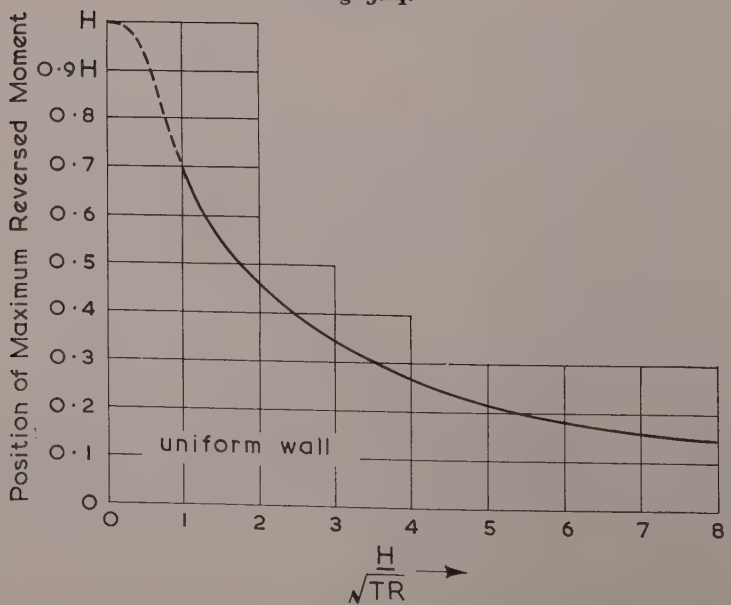


Fig. 3.15.

curve is due to the fact that it consists of two separate curves. Up to a value of $H/\sqrt{TR} = 1{\cdot}76$, the maximum hoop tension occurs at the top of the wall (see *Fig.* 3.3). Above this value the top of the wall moves inwards and the point of maximum tension moves further and further down the wall (see *Figs.* 3.4, 3.5 and 3.6). In *Fig.* 3.17 the lower part of *Fig.* 3.16 is shown to a larger scale and the position of the maximum hoop tension is shown in *Fig.* 3.18.

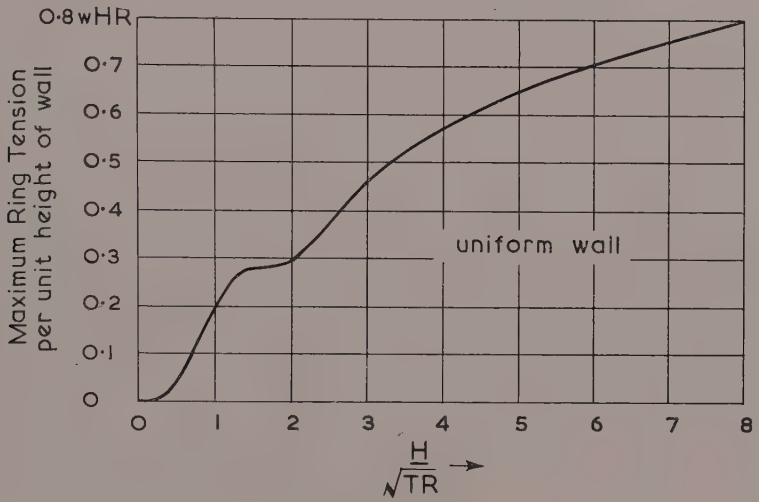


Fig. 3.16.

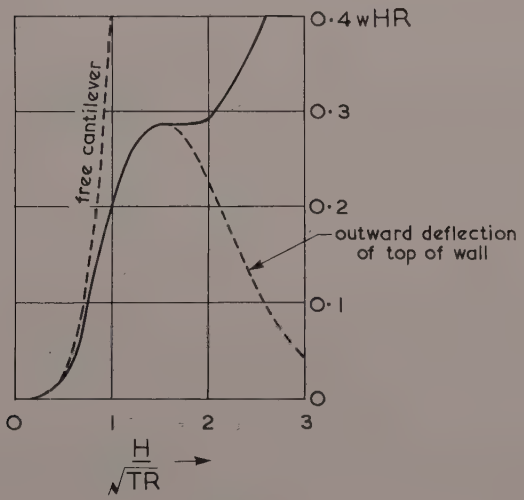


Fig. 3.17.

Although the principal values of the moments, shears and hoop tensions may be read approximately from *Figs.* 3.3 to 3.18, for a large tank, the reader should solve the fundamental equations for himself. To facilitate this, *Figs.* 3.19 and 3.20 give the values of the coefficients A and B for values of $H/\sqrt{TR}$ up to 4·0.

All tabulated values are accurate only to within the limits of a slide-rule.

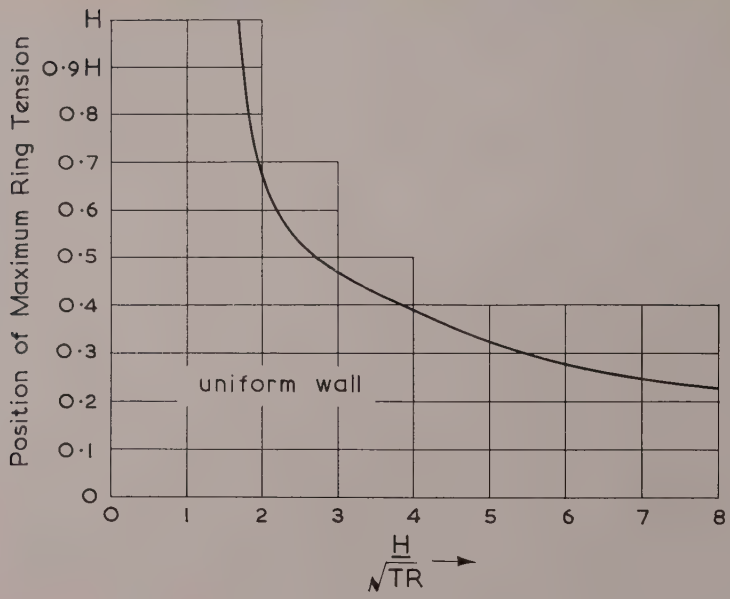


Fig. 3.18.

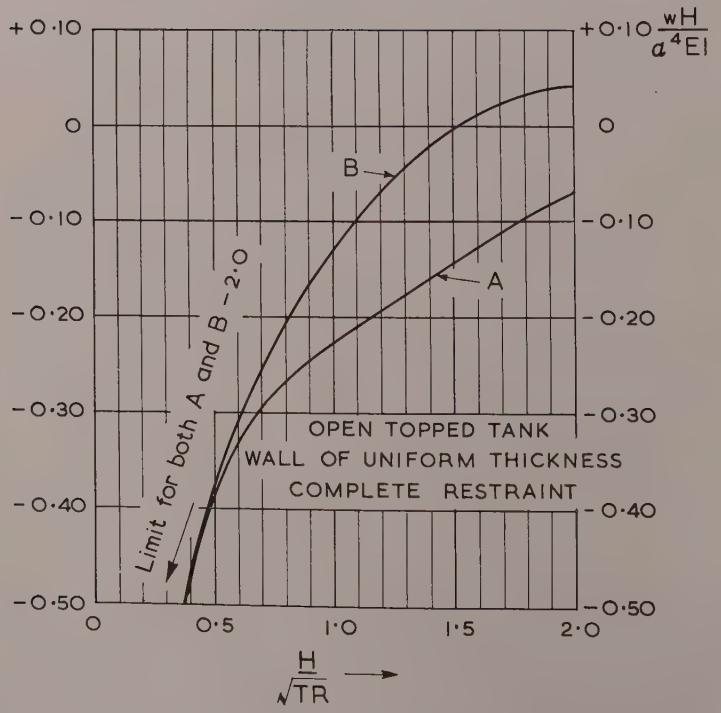


Fig. 3.19.

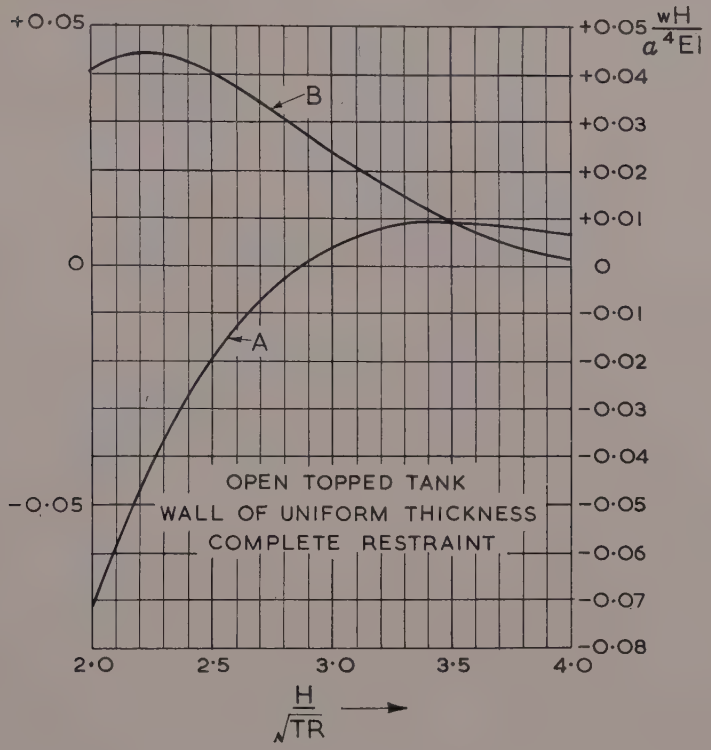


Fig. 3.20.

Units of Measurement

If H is the depth of the tank in feet, T the thickness of the wall in feet, R the radius of the tank in feet, and w the weight of the contents in pounds per cu. ft., then the moments are in pounds-feet per foot run of perimeter, the shear force is in pounds per foot run of perimeter and the ring tension is in pounds per foot height of wall.

Restraint under Earth Pressure

If earth is banked up round the outside of the tank up to water level and if the earth pressure at any given depth h below water level is kh then, when the tank is empty, reversed restraint effects are obtained, which can be determined at once from the curves by writing k instead of w. In former times some designers would rely on the external earth pressure to reduce the stresses in the tank when it is full of water. Modern specifications generally insist that the tank should be filled and tested before any earth banking is placed.

Wall Thickness in Relation to CP.2007:1960

According to CP.2007:1960, two conditions must be satisfied regarding the prevention of cracking. Firstly, the concrete stress in bending must not exceed about 300 lb. per sq. in. in tension. This is governed by the restraint moment

M_B at the base of the wall (*Figs.* 3.12 and 3.13). Secondly the concrete stress in hoop tension must not exceed about 200 lb. per sq. in. (*Fig.* 3.16). The former condition will be found to be the more severe.

For example, if $H/\sqrt{TR} = 3$, $M_B = 0{\cdot}216wHTR$ (*Fig.* 3.13) and maximum hoop tension $= 0{\cdot}46wHR$ (*Fig.* 3.16).

If the safe moment of resistance is $50bd^2$ in inch units and if w, H, T and R are all in pounds and foot units,

$$50bd^2 = 50 \times 12 \times 144T^2 = 0{\cdot}216wHTR \times 12 \text{ and}$$
$$T \text{ (to resist } M_B) = 0{\cdot}00003wHR.$$

At a safe tension of 200 lb. per sq. in.,

$$T \text{ (to resist maximum hoop tension)} = \frac{0{\cdot}46wHR}{200 \times 12 \times 12} = 0{\cdot}000016wHR.$$

The minimum thickness of wall is therefore generally governed by M_B. But before M_B can be found from *Figs.* 3.12 and 3.13, a value must be assumed for T. If

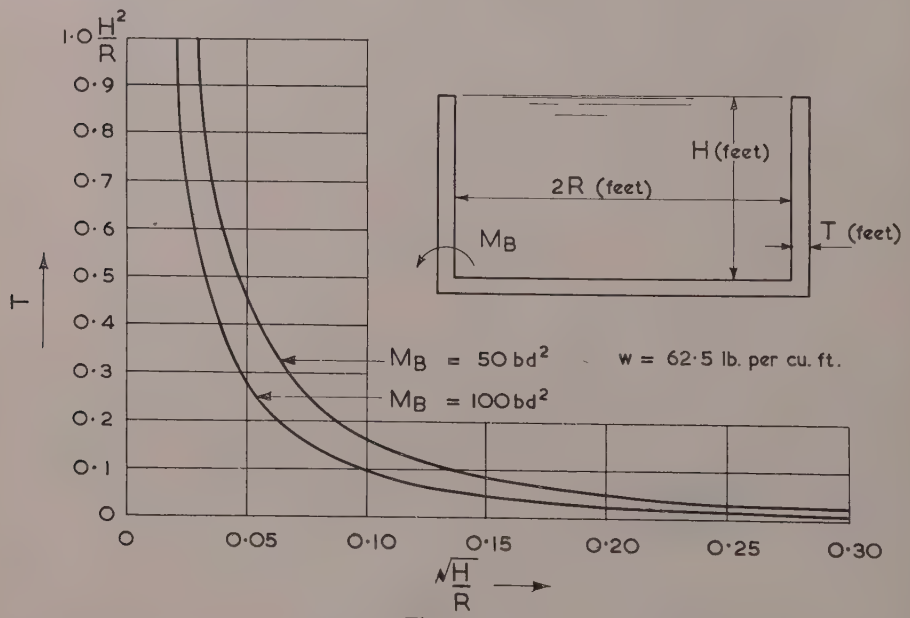


Fig. 3.21.

the first assumption is too small and a larger value is necessary, the moment M_B will also increase.

Usually H and R are known and T is required. If the values of w and M/bd^2 are fixed a curve can be plotted showing the relation between $\sqrt{H/R}$ and T in terms of H^2/R. Assuming $w = 62{\cdot}5$ lb. per cu. ft. and $M/bd^2 = 50$, for $H/\sqrt{TR} = 3$,

$$T = 0{\cdot}00003wHR \text{ (as before)} = 0{\cdot}00188HR.$$

Now $$H^2 = 9TR \text{ or } T = 0{\cdot}111H^2/R.$$

Therefore $$0{\cdot}00188HR = 0{\cdot}111H/R^2 \text{ or } H/R^2 = 0{\cdot}0169 \text{ and } \sqrt{H/R} = 0{\cdot}130.$$

With these loads and stresses, when $\sqrt{H/R} = 0{\cdot}130$, $T = 0{\cdot}111H^2/R$; this gives

one point on *Fig.* 3.21. This figure also shows the value of T if $w = 62{\cdot}5$ lb. per cu. ft. and $M/bd^2 = 100$. It will be seen that if the value of M/bd^2 is increased from 50 to 100 the value of T for any given value of $\sqrt{H/R}$ is reduced by about 40 per cent., thus saving 40 per cent. of the concrete in the wall. Now the effect of M_B is extremely local, the moment reducing very quickly away from the foot of the wall.

When forced to comply with requirements of CP.2007:1960 it is generally cheaper to reinforce the bottom corner very heavily with short bars, making the value of M/bd^2 as high as is reasonable. With a ratio $H/\sqrt{TR} = 3$ the restraint moment dies away completely before a height of $0{\cdot}2H$ above the floor is reached.

Moreover, extra steel is often available from the floor. If the floor is made the same thickness as the base of the wall the section of floor immediately adjoining the base of the wall has not only to resist the restraint moment M_B but may also have to carry direct tension from the restraint shear F_B. If all the reinforcement in the floor is turned up into the wall it will considerably raise the value of M/bd^2.

The reversed moment is easily dealt with as the Code permits the use of much higher stresses in this face.

Partial Restraint

Where the site consists of some 30 ft. or 40 ft. of very soft ground overlying a much harder stratum the tank will normally be supported on piles. In general the piles will all be vertical and the restraint shear at the base of the wall is taken in tension across the floor of the tank. The floor stretches and the bottom of the wall moves outwards reducing the restraint. Even if raking piles are driven to take the horizontal load the heads of these piles must move outwards before they can take up their full load. In the general equation

$$y = \frac{wh}{\alpha^4 EI} + \left[e^{\frac{\alpha h}{\sqrt{2}}}\left(A \sin \frac{\alpha h}{\sqrt{2}} + B \cos \frac{\alpha h}{\sqrt{2}} \right) + e^{-\frac{\alpha h}{\sqrt{2}}}\left(C \sin \frac{\alpha h}{\sqrt{2}} + D \cos \frac{\alpha h}{\sqrt{2}} \right) \right] \quad (1)$$

With full restraint, $y = 0$ when $h = H$ or

$$0 = \frac{wH}{\alpha^4 EI} + \left[e^{\frac{\alpha H}{\sqrt{2}}}\left(A \sin \frac{\alpha H}{\sqrt{2}} + B \cos \frac{\alpha H}{\sqrt{2}} \right) + e^{-\frac{\alpha H}{\sqrt{2}}}\left(C \sin \frac{\alpha H}{\sqrt{2}} + D \cos \frac{\alpha H}{\sqrt{2}} \right) \right].$$

where $$\alpha^4 = \frac{T}{R^2 I}$$

and $$\frac{wH}{\alpha^4 EI} = \frac{wHR^2}{ET}.$$

Now the expression wHR^2/ET is the amount by which the bottom end of the wall would move outwards if it had a frictionless sliding joint (*Fig.* 3.22) and the term $wh/\alpha^4 EI$ in equation (1) is the equation of the 'free-stretch' line AC and the term in the square brackets is the amount by which the 'SHAPE' curve of the wall departs from this line AC.

If the restraint shear is F_B and the thickness of the tank floor is T_F, then the tensile stress in the floor is F_B/T_F and the edge of the floor will stretch outwards by an amount $\frac{F_B}{T_F} \times \frac{R}{E}$. If this is expressed as a fraction of wHR^2/ET the arithmetic is simplified.

If $\frac{F_B R}{T_F E} = 0{\cdot}25\,\frac{wHR^2}{ET}$, as shown in *Fig.* 3.22, then point B moves out to B′, where BB′ is equal to 0·25 BC and the restraint is only 75 per cent. effective; hence $y = 0{\cdot}25wHR^2/ET$ when $h = H$, or

$$0{\cdot}25\frac{wHR^2}{ET} = \frac{wHR^2}{ET} + \left[e^{\frac{\alpha H}{\sqrt{2}}}\left(A\,\sin\frac{\alpha H}{\sqrt{2}} \ldots\right)\right]$$

$$-\,0{\cdot}75\,\frac{wHR^2}{ET} = \left[e^{\frac{\alpha H}{\sqrt{2}}}\left(A\,\sin\frac{\alpha H}{\sqrt{2}} \ldots\right)\right]$$

or

$$-\,0{\cdot}75\,\frac{wH}{\alpha^4 EI} = \left[e^{\frac{\alpha H}{\sqrt{2}}}\left(A\,\sin\frac{\alpha H}{\sqrt{2}} \ldots\right)\right].$$

If $H/\sqrt{TR} = 2$ and the lateral restraint is only 75 per cent. effective, the deformed shape of the wall is as shown in *Fig.* 3.22. This figure also shows the shape if the restraint is complete. In *Fig.* 3.23 the corresponding moments in the vertical ' staves ' for both conditions are shown. The moment at the base of the wall falls

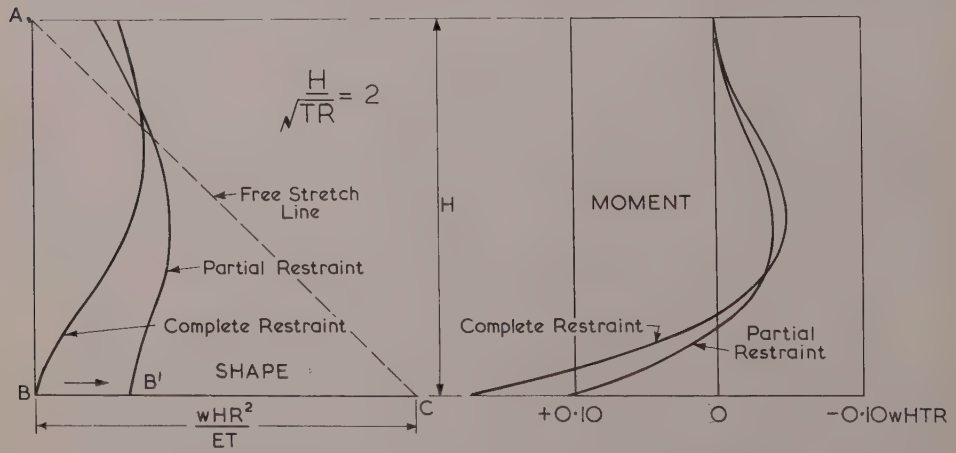


Fig. 3.22. **Fig. 3.23.**

from $0{\cdot}1715wHTR$ to $0{\cdot}1015wHTR$ (a reduction of 40 per cent. in the moment for a loss of 25 per cent. of the restraint) and the restraint shear falls from $0{\cdot}2975wH^2$ to $0{\cdot}205wH^2$ (a reduction of 30 per cent.). The hoop tensions in the middle and bottom sections of the wall are increased but the hoop tension at the top is decreased. As the restraint is further relaxed the wall tends to rock over more and more towards the ' free-stretch ' line AC.

If $H/\sqrt{TR} = 3$ and the base of the wall moves out $0{\cdot}5wHR^2/ET$ (the tangent at the base of the wall remaining vertical) the restraint moment at the base of the wall decreases from $0{\cdot}216wHTR$ to $0{\cdot}0715wHTR$ (a reduction of 67 per cent.) and the restraint shear decreases from $0{\cdot}222wH^2$ to $0{\cdot}0948wH^2$ (a reduction of 57 per cent.).

When the release of restraint is due to stretching of the floor the effect tends to cancel itself out. As the floor stretches the restraint shear falls and the tension in the floor decreases, until equilibrium is reached. This may mean one or two approximations must be tried before a sufficiently accurate answer is obtained, as shown by the following example.

EXAMPLE:—A circular tank to hold sewage weighing 68 lb. per cu. ft. is 85 ft. in diameter with an effective depth of 27 ft. 8 in. The tank is supported on vertical piles, the wall and floor both being 1 ft. 6 in. in thickness. A simplified version of the tank is shown in *Fig.* 3.24, the actual tank having small drainage slopes,

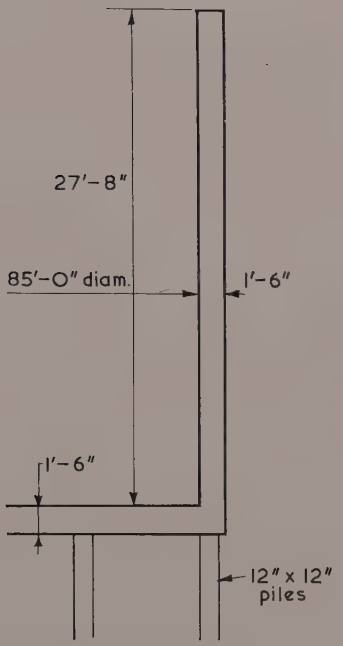


Fig. 3.24.

recesses in the floor, loads on cantilevered brackets on the wall, etc. which complicate the details but add nothing to the main problem of restraint.

To obtain a first approximation the analysis is first carried out assuming complete restraint.

$$\frac{H}{\sqrt{TR}} = \frac{27{\cdot}67}{\sqrt{1{\cdot}5 \times 42{\cdot}5}} = 3{\cdot}46 \text{ and as } \alpha \text{ is always equal to } \frac{1{\cdot}86}{\sqrt{TR}},$$

$$\frac{\alpha H}{\sqrt{2}} = \frac{1{\cdot}86}{\sqrt{TR}} \times \frac{H}{\sqrt{2}} = \frac{1{\cdot}86 \times 3{\cdot}46}{\sqrt{2}} = 4{\cdot}55 \text{ radians.}$$

$$e^{4{\cdot}55} = 94{\cdot}632. \quad \sin 4{\cdot}55 \text{ rad.} = -0{\cdot}987.$$
$$e^{-4{\cdot}55} = 0{\cdot}01057. \quad \cos 4{\cdot}55 \text{ rad.} = -0{\cdot}00171.$$

Putting $y = 0$ when $h = H$, and remembering that $A = C$ and $D = B - 2A$ (since this is an open-topped tank),

$$-93{\cdot}41A - 15{\cdot}31B = -1{\cdot}0\frac{wH}{\alpha^4 EI} \quad . \quad . \quad . \quad \text{(a)}$$

Also since $\frac{dy}{dh} = 0$ when $h = H$, from equation (8)

$$\frac{\alpha}{\sqrt{2}}(-108{\cdot}68A + 78{\cdot}10B) = -\frac{w}{\alpha^4 EI}.$$

As $$\alpha = \frac{1{\cdot}86}{\sqrt{TR}} \text{ and } \frac{H}{\sqrt{TR}} = 3{\cdot}46, \; \alpha = \frac{1{\cdot}86 \times 3{\cdot}46}{H}.$$

$$-108{\cdot}68A + 78{\cdot}10B = -0{\cdot}2198\frac{wH}{\alpha^4 EI} \quad . \quad . \quad . \quad (b)$$

Solving equations (a) and (b),

$$A = +0{\cdot}00909\frac{wH}{\alpha^4 EI} \text{ and } B = +0{\cdot}00984\frac{wH}{\alpha^4 EI}$$

As $C = A$ and $D = B - 2A$ all four coefficients for this particular tank under these particular conditions have now been determined.

Substituting these coefficients in the equation for y the deflected shape of the wall is obtained (*Fig.* 3.25). Resubstituting in the equation for $\frac{d^2y}{dh^2}$ the moments are obtained (*Fig.* 3.26).

The restraint shear is $EI\frac{d^3y}{dh^3}$ when $h = H$ and is

$$0{\cdot}676wH\sqrt{TR} = 0{\cdot}1955wH^2 = 0{\cdot}1955 \times 68 \times 27{\cdot}67^2$$
$$= 10{,}200 \text{ lb. per foot run of wall.}$$

The tension in the floor is $\frac{10{,}200}{18 \times 12} = 47{\cdot}2$ lb. per sq. in.

The ' free-stretch ' line AC reaches a maximum value BC which corresponds to a ring tension of $\frac{wHR}{T} = \frac{68 \times 27{\cdot}67 \times 42{\cdot}5}{1{\cdot}5} = 53{,}3000$ lb. per sq. ft. $= 370$ lb. per sq. in. Since $\frac{47{\cdot}2}{370} = 0{\cdot}128$, the extension of the floor under the tension induced by the restraint shear allows the bottom point B of the wall to move out 12·8 per cent. of the distance it would move if it had a frictionless sliding joint. This value of 12·8 per cent. is the first approximation and is now used to obtain a second approximation.

$$\frac{F_B}{T_F} \times \frac{R}{E} = \frac{10{,}200}{1{\cdot}5} \times \frac{R}{E}$$

and $$\frac{wHR^2}{ET} = \frac{68 \times 27{\cdot}67 \times 42{\cdot}5R}{E \times 1{\cdot}5}.$$

So $$\frac{F_B}{T_F} \times \frac{R}{E} = \frac{10{,}200}{1{\cdot}5} \times \frac{1{\cdot}5wHR^2}{68 \times 27{\cdot}67 \times 42{\cdot}5 \times ET} = 0{\cdot}128\frac{wHR^2}{ET} = 0{\cdot}128\frac{wH}{\alpha^4 EI}.$$

With a tension of 47·2 lb. per sq. in. in the floor, the lateral restraint is only (1·0 − 0·128) or 87·2 per cent. effective.

When $h = H$, therefore $y = 0{\cdot}128\frac{wH}{\alpha^4 EI}$ so equation (a) now becomes

$$-93{\cdot}41A - 15{\cdot}31B = -\frac{1{\cdot}0wH}{\alpha^4 EI} + 0{\cdot}128\frac{wH}{\alpha^4 EI} \quad . \quad . \quad (a_1)$$

Assuming that the corner does not rotate, equation (b) still holds good. Solving the equations,

$$A = +0{\cdot}00797\frac{wH}{\alpha^4 EI} \text{ and } B = +0{\cdot}00829\frac{wH}{\alpha^4 EI},$$

and this modifies the restraint shear value, for $EI\frac{d^3y}{dh^3}$ when $h = H$ is now equal to $0{\cdot}579wH\sqrt{TR} = 0{\cdot}1673wH^2 = 8{,}700$ lb. only. This is a second approximation against the first of 10,200 lb. The tension in the floor due to the shear of 8,700 lb. is 40·3 lb. per sq. in., corresponding to an extension of only 10·9 per cent., against the first approximation of 12·8 per cent. Interpolating between these results a final extension of 11·2 per cent. is obtained, corresponding to a tension of 41·3 lb. per sq. in. in the floor.

Equation (a) now becomes:

$$-93{\cdot}41A - 15{\cdot}31B = -1{\cdot}0\frac{wH}{\alpha^4 EI} + 0{\cdot}112\frac{wH}{\alpha^4 EI} \quad . \quad . \quad (a_2)$$

Assuming once again that the tangent at the base of the wall remains vertical, equation (b) still applies and finally

$$A = +0{\cdot}00811\frac{wH}{\alpha^4 EI} \text{ and } B = +0{\cdot}00848\frac{wH}{\alpha^4 EI}.$$

Resubstituting these values in the expressions for y and $\frac{d^2y}{dx^2}$ the results shown in *Figs.* 3.25 and 3.26 are obtained.

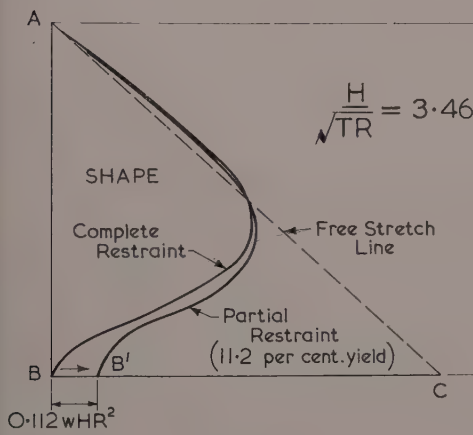


Fig. 3.25.

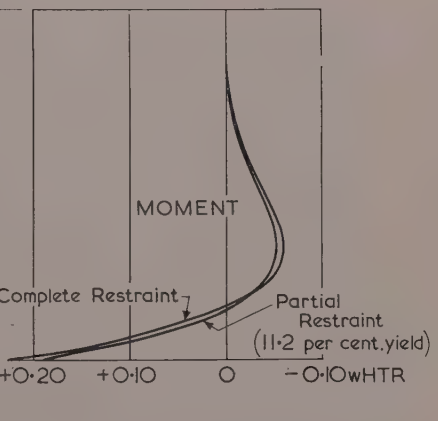


Fig. 3.26.

The extension of the floor has reduced the restraint moment from $0{\cdot}2252wHTR$ to $0{\cdot}1930wHTR$ (a reduction of 14 per cent.) but the maximum value of y (and the corresponding maximum hoop tension) increases only from 0·52 to $0{\cdot}53\frac{wHR^2}{ET}$ (an increase of only 2 per cent.).

The actual reinforcement provided is shown in *Fig.* 3.27. The wall is 18 in. in thickness and the vertical bars near the inside face at the base of the wall are 1 in. in diameter at 6-in. centres and near the outside face $\frac{3}{4}$ in. in diameter at 1-ft. 6-in. centres. The tank was designed for steel stresses of 16,000 and 13,500 lb. per sq. in. in bending and in hoop tension, respectively. The tensile stress in the concrete in bending was not limited but in hoop tension in the concrete alone it was not to exceed 300 lb. per sq. in.

For the section at the base of the wall $b = 12$ in., $d = 18$ in., $d_1 = 16$ in., $A_{st} = 1{\cdot}57$ sq. in., and $A_{sc} = 0{\cdot}294$ sq. in.

The moment $= 0{\cdot}1930wHTR$

$= 0{\cdot}1930 \times 68 \times 27{\cdot}67 \times 1{\cdot}5 \times 42{\cdot}5$ lb.-ft. per foot run

$= 23{,}150$ lb.-ft. $= 278{,}000$ lb.-in.

Neglecting the steel, the maximum stresses on the concrete alone are

$$\pm \frac{278{,}000 \times 6}{12 \times 18^2} = \pm 430 \text{ lb. per sq. in.}$$

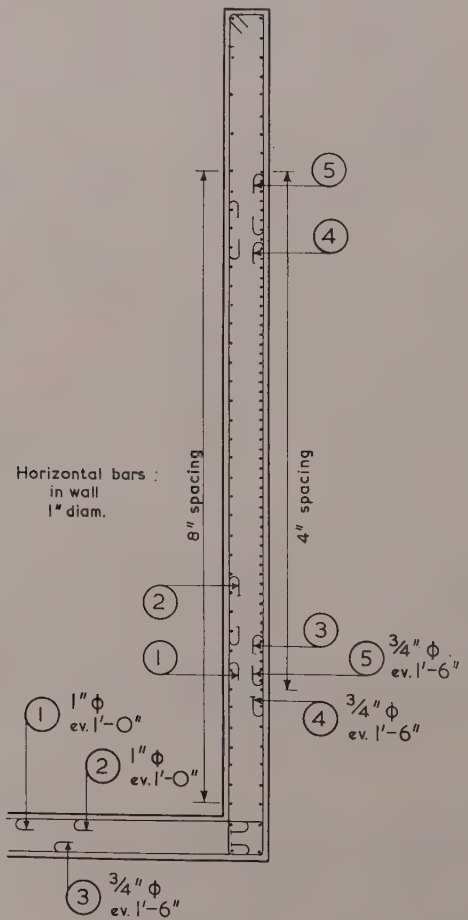


Fig. 3.27.

The actual weight of the wall plus imposed loads is about 8,000 lb. per foot run causing a stress of $8{,}000/(12 \times 18) = +37$ lb. per sq. in., and giving total stresses of from $+467$ to -393 lb. per sq. in.

The tensile stress in the steel, neglecting the tensile stresses in the concrete, is

$$= \frac{278{,}000}{1{\cdot}57 \times 0{\cdot}9 \times 16} = 12{,}300 \text{ lb. per sq. in.}$$

Taking the combined section with steel and concrete acting together the maximum stresses are +412 to −296 lb. per sq. in.

The maximum value of y is $0{\cdot}53\frac{wH}{\alpha^4 EI}$. The maximum hoop tension is therefore 0·53 times the hoop tension at the base of a free-stretching wall

$$= 0{\cdot}53 \times 68 \times 27{\cdot}67 \times 42{\cdot}5$$
$$= 42{,}300 \text{ lb. per foot of height.}$$

At the point of maximum hoop tension the wall is reinforced with 1-in. diam. bars at 4-in. centres near the outside face and with 1-in. diam. bars at 8-in. centres near the inside face.

Total steel area provided = 2·36 + 1·18 = 3·54 sq in.
Tensile stress in concrete alone = 42,300/(12 × 18) = 196 lb. per sq. in.
Tensile stress in steel alone = 42,300/3·54 = 12,000 lb. per sq. in.

$$\text{Tensile stress on combined section} = \frac{42{,}300}{(12 \times 18) + (14 \times 3{\cdot}54)} = 159 \text{ lb. per sq. in.}$$

The maximum reversed moment is $-0{\cdot}054wHTR = -77{,}800$ lb.-in.
The reinforcement provided consists of $\frac{3}{4}$-in. diam. bars at 9-in. centres = 0·59 sq. in. per foot run.
Tensile stress in steel, neglecting tensile stresses in concrete,

$$= \frac{77{,}800}{0{\cdot}59 \times 0{\cdot}9 \times 16} = 9{,}200 \text{ lb. per sq. in.}$$

There are altogether 104 1-in. diam. bars encircling the tank. The total hoop tension, assuming the joint at the bottom of the wall were to fail, would be

$$\tfrac{1}{2} \times 68 \times 27{\cdot}67^2 \times 42{\cdot}5 = 1{,}110{,}000 \text{ lb.}$$

The average stress in the hoop steel would be $\frac{1{,}110{,}000}{104 \times 0{\cdot}785} = 13{,}500$ lb. per sq. in.

As in all practical design the actual arrangement of bars was arrived at after an examination of the site and after careful consideration of the conditions under which the tank would have to work.

Loss of Angular Restraint

So far only the loss of lateral restraint has been considered, but the restraint moment also tends to twist the tangent at the base of the wall outwards. This increases the intensity of pressure on the ground under the extreme edge of the floor and unless the ground is really hard (or unless the tank is supported on sufficient piles) some relative settlement may occur and the corner may actually rotate.

The general equation for the tangent slope is

$$\frac{dy}{dh} = \frac{w}{\alpha^4 EI} + \frac{\alpha}{\sqrt{2}}\left[e^{\frac{\alpha h}{\sqrt{2}}}\left(A \sin\frac{\alpha h}{\sqrt{2}} + B\cos\frac{\alpha h}{\sqrt{2}} + A\cos\frac{\alpha h}{\sqrt{2}} - B\sin\frac{\alpha h}{\sqrt{2}}\right)\right.$$
$$\left. + e^{-\frac{\alpha h}{\sqrt{2}}}\left(-C\sin\frac{\alpha h}{\sqrt{2}} - D\cos\frac{\alpha h}{\sqrt{2}} + C\cos\frac{\alpha h}{\sqrt{2}} - D\sin\frac{\alpha h}{\sqrt{2}}\right)\right] \quad . \qquad (8)$$

The term $\frac{w}{\alpha^4 EI}$ is equal to $\frac{wR^2}{ET}$, since $\alpha = \sqrt[4]{\frac{T}{R^2 I}}$.

Now the free stretch at the bottom of a wall with a sliding joint is wHR^2/ET and the inclination of the 'free-stretch' line AC to the vertical is

$$\frac{wHR^2}{ET} \times \frac{1}{H} = \frac{wR^2}{ET} = \frac{w}{\alpha^4 EI},$$

which is the first term in equation (8). If the loss of angular restraint can be estimated, calculation will be avoided if the rotation of the bottom corner of the wall is expressed in terms of $w/\alpha^4 EI$.

Again considering the case of a wall with the ratio $H/\sqrt{TR} = 3{\cdot}46$ (the wall shown in *Fig.* 3.24) and assuming that the joint between wall and floor is hinged but remains in its original position, there is complete lateral restraint but a complete loss of angular restraint.

When $h = H$, instead of $\frac{dy}{dh} = 0$, now $M = 0$ and as $M = EI\frac{d^2y}{dh^2}$ then $\frac{d^2y}{dh^2} = 0$.

$$0 = \alpha^2\left[e^{\frac{\alpha H}{\sqrt{2}}}\left(A \cos \frac{\alpha H}{\sqrt{2}} - B \sin \frac{\alpha H}{\sqrt{2}}\right) + e^{-\frac{\alpha H}{\sqrt{2}}}\left(-C \cos \frac{\alpha H}{\sqrt{2}} + D \sin \frac{\alpha H}{\sqrt{2}}\right)\right]$$

or

$$-15{\cdot}287A + 93{\cdot}39B = 0.$$

Since equation (a) still holds good,

$$A = +0{\cdot}01043\frac{wH}{\alpha^4 EI} \text{ and } B = +0{\cdot}001706\frac{wH}{\alpha^4 EI}.$$

Resubstituting these values in the original equations the deflected shape of the wall and the moments induced are as shown in *Figs.* 3.28 and 3.29.

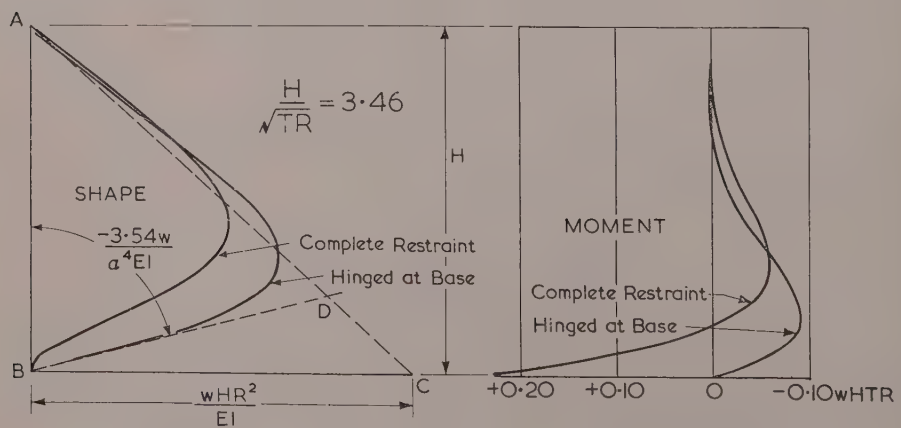


Fig. 3.28. **Fig. 3.29.**

When $h = H$ the value of $\frac{dy}{dh}$ is found from equation (8).

$$\frac{dy}{dh} = \frac{w}{\alpha^4 EI} + \frac{\alpha}{\sqrt{2}}[-108{\cdot}68A + 78{\cdot}10B] = -3{\cdot}54\frac{w}{\alpha^4 EI}.$$

This is three-and-a-half times the slope of the 'free stretch' line AC. In *Fig.* 3.28 the angle ABD is 3·54 times the angle BAC. The deflected shape of and moments induced in a fully-restrained wall are also shown in *Figs.* 3.28 and 3.29.

If therefore the bottom corner of the wall of a tank with the ratio $H/\sqrt{TR} = 3{\cdot}46$ rocks outwards by an angle $-3{\cdot}54\frac{w}{\alpha^4 EI}$ the restraint moment vanishes.

Now $-3{\cdot}54\frac{w}{\alpha^4 EI} = -3{\cdot}54\frac{wR^2}{ET}$ and if $E = 3{,}000{,}000 \times 144$ lb. per sq. ft., then

$$-3{\cdot}54\frac{wR^2}{ET} = -\frac{3{\cdot}54 \times 68 \times 42{\cdot}5^2}{3{,}000{,}000 \times 144 \times 1{\cdot}5} = 0{\cdot}000672 \text{ radians} = 0{\cdot}0384 \text{ deg.,}$$

or a slope of 0·223 in. in a height of 27 ft. 8 in. This result again emphasizes the very small movements required to upset the assumption of complete restraint, and indicates the grossly exaggerated horizontal scale to which the 'SHAPE' curves are drawn.

CHAPTER FOUR

TOP RESTRAINT IN CIRCULAR TANKS

BEFORE discussing any practical aspects of the case where the top of the wall of a circular tank is restrained, the moments and tensions will be calculated on the assumption that the top end of the wall is absolutely fixed in position and direction at water level.

The conditions of loading remain the same as for an open-topped tank so that

$$y = \frac{wh}{\alpha^4 EI} + \left[e^{\frac{\alpha h}{\sqrt{2}}} \left(A \sin \frac{\alpha h}{\sqrt{2}} + B \cos \frac{\alpha h}{\sqrt{2}} \right) + e^{-\frac{\alpha h}{\sqrt{2}}} \left(C \sin \frac{\alpha h}{\sqrt{2}} + D \cos \frac{\alpha h}{\sqrt{2}} \right) \right] . \quad (1)$$

where $$\alpha = \sqrt[4]{\frac{T}{R^2 I}} = \frac{1{\cdot}86}{\sqrt{TR}}.$$

However, the end conditions at the top are different. If the wall is absolutely restrained in position and direction at the top, $y = 0$ when $h = 0$ giving

$$B + D = 0 \quad . \quad . \quad . \quad . \quad . \quad (14)$$

$\frac{dy}{dh} = 0$ when $h = 0$ giving

$$0 = \frac{w}{\alpha^4 EI} + \frac{\alpha}{\sqrt{2}}[B + A - D + C].$$

Since $B + D = 0$,

$$A + 2B + C = -\frac{w}{\alpha^4 EI} \times \frac{\sqrt{2}}{\alpha} \quad . \quad . \quad . \quad (15)$$

Also $$y = 0 \quad \text{when} \quad h = H \; . \quad . \quad . \quad . \quad . \quad (4)$$

and $$\frac{dy}{dh} = 0 \quad \text{when} \quad h = H \; . \quad . \quad . \quad . \quad . \quad (5)$$

For any given ratio of $H/\sqrt{TR}$, $\alpha H/\sqrt{2}$ can at once be calculated to obtain the numerical coefficients in equations (4) and (5), and from these, together with equations (14) and (15), values for A, B, C and D, may be obtained.

For example if $H/\sqrt{TR} = 1{\cdot}0$, then $H = \sqrt{TR}$.

$$\frac{\alpha H}{\sqrt{2}} = \frac{1{\cdot}86}{\sqrt{TR}} \times \frac{H}{\sqrt{2}} = \frac{1{\cdot}86}{\sqrt{2}} = 1{\cdot}315.$$

$e^{1{\cdot}315} = 3{\cdot}725$, $e^{-1{\cdot}315} = 0{\cdot}2685$, sin 1·315 rad. = 0·9675 and cos 1·315 rad. = 0·2530. Substituting these values in equations (1) and (8) and using equations (14) and (15) and solving:

$$A = -0{\cdot}2301 \frac{wH}{\alpha^4 EI}; \quad B = -0{\cdot}0945 \frac{wH}{\alpha^4 EI};$$

$$C = -0{\cdot}3410 \frac{wH}{\alpha^4 EI}; \quad D = +0{\cdot}0945 \frac{wH}{\alpha^4 EI}.$$

Resubstituting these values in equation (1) the deflected shape of the wall is as shown

in *Fig.* 4.1. This figure also shows the shape of a similar wall for an open-topped tank. As might be expected since the open-topped wall behaves in a manner so similar to that of a pure cantilever, the behaviour of the fixed-top wall is even more

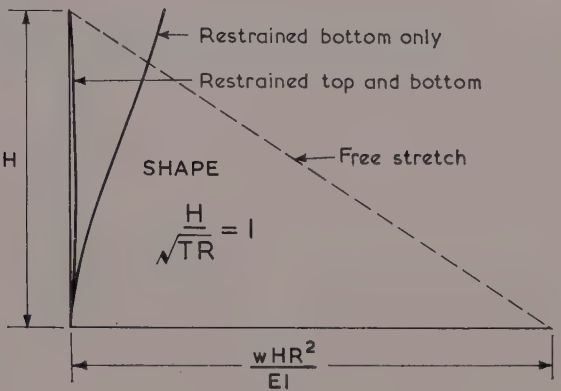


Fig. 4.1.

similar to that of a vertical beam fixed top and bottom. The maximum value of y is $0{\cdot}012wHR^2/ET$, or less than 2 per cent. of the extension at the base of a totally unrestrained wall. The moments, calculated from equation (9), are

$$M_A = +0{\cdot}032wH^3 \quad \text{and} \quad M_B = +0{\cdot}0496wH^3.$$

For a pure beam without support from hoop tension the corresponding figures are $M_A = +0{\cdot}033wH^3$ and $M_B = +0{\cdot}050wH^3$. The walls of all circular tanks with ratios of $H/\sqrt{TR}$ of less than unity, which are restrained at both top and bottom may therefore be designed as vertical beams fixed at both ends. In *Fig.* 4.2 the

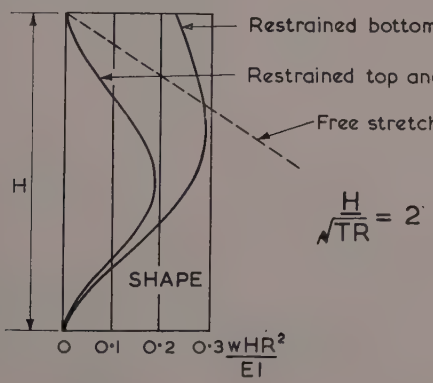


Fig. 4.2.

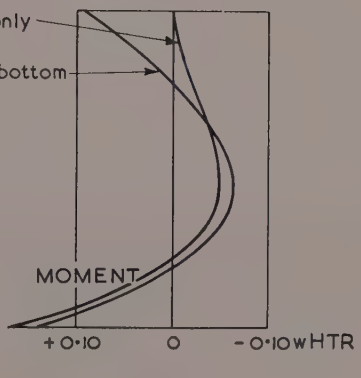


Fig. 4.3.

deflected shape of the wall if $H/\sqrt{TR} = 2$ is shown and *Fig.* 4.3 shows the moments induced in the vertical 'staves'. Both figures also indicate the corresponding values for an open-topped tank.

The deflected shape of the wall when $H/\sqrt{TR} = 3$ is shown in *Fig.* 4.4, and *Fig.* 4.5 shows the moments induced in this case.

Fixing the wall at the top will affect the moments and hoop tensions throughout

the entire height of the wall, as is clearly shown in *Figs.* 4.2, 4.3, 4.4 and 4.5 but, as the value of $H/\sqrt{TR}$ increases, the effect of the top restraint becomes more and more local and only the moment and shear force at the top end need be known for

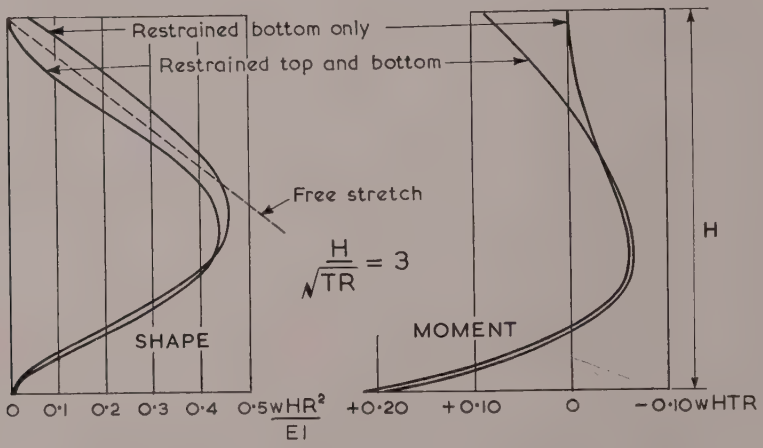


Fig. 4.4.

Fig. 4.5.

walls whose ratio of $H/\sqrt{TR}$ exceeds 3. The values of the moments and shears are as shown in *Figs.* 4.6, 4.7 and 4.8. The values of the moment shown in *Figs.* 4.6 and 4.7 are the same, but they are expressed in different terms.

Practical Application

The assumption that the top edge of the wall may be fixed exactly in position and direction can scarcely be realized in practice. The assumption that it is fixed at water-level is seldom true as there is usually a small freeboard between the water-

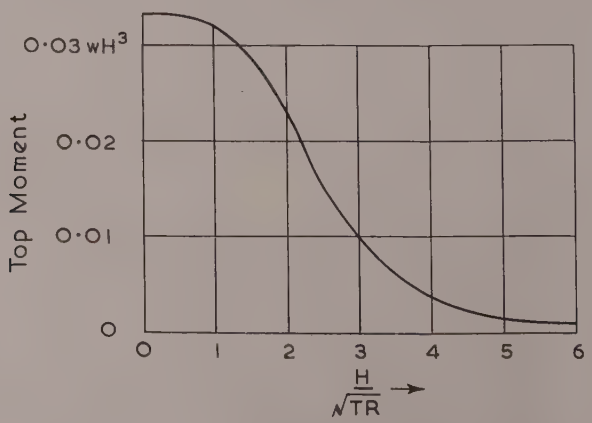


Fig. 4.6.

level and the underside of the roof. Most tanks contain water, and the roof, not being actually wet, may shrink a little relative to the wall; this would tend to intensify the top restraint. Unless the roof is protected by effective insulation it is sure to be hotter than the walls on a hot sunny day, and the consequent expansion could cause

severe reverse restraint effects. A hoop tension of 300 lb. per sq. in. in the wall increases the radius by $\frac{300R}{3{,}000{,}000}$ or $0{\cdot}0001R$. A relative difference of 45 deg. F. (25 deg. C.) means a difference of $45 \times 0{\cdot}000006R$ or $0{\cdot}00027R$. In such circumstances it might be best to avoid all structural connection between wall and roof and insert a layer of free-sliding synthetic material between them (in which case, of course, there is no restraint effect).

The assumption that the roof is sufficiently stiff to keep the edge tangent in its original position is most unlikely to be realized. A flexible roof loaded heavily will deflect downwards at midspan and could impose a large reversed moment on the

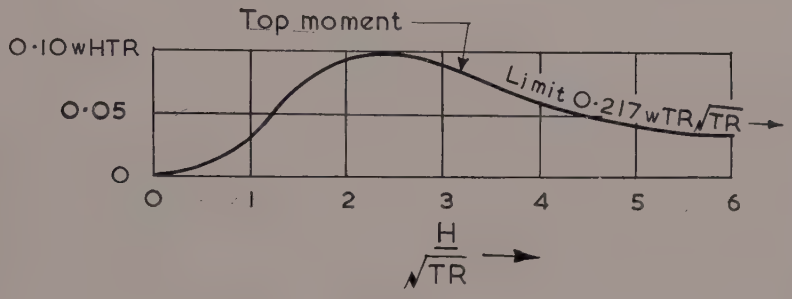


Fig. 4.7.

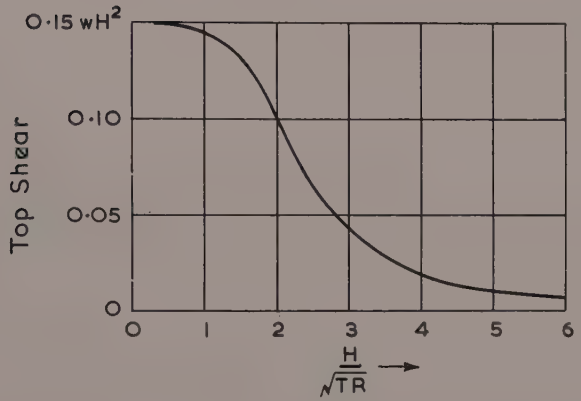


Fig. 4.8.

top of the wall, exceeding the normal restraint moment. The unit moment at the edge of a flat circular plate fixed round its periphery is $\frac{1}{8}wR^2$.

As an example consider the case of the tank of a water tower 25 ft. in diameter and 15 ft. deep, with a 6-in. wall and a roof slab 9 in. in thickness.

$$\frac{H}{\sqrt{TR}} = \frac{15}{\sqrt{0{\cdot}5 \times 12{\cdot}5}} = 6.$$

The total load on the roof including screed, asphalt, snow load and self-weight is 160 lb. per sq. ft.

The restraint moment, assuming the roof is absolutely rigid and the wall is relatively flexible, (from *Fig.* 4.7) is $0{\cdot}036wHTR$

$= 0{\cdot}036 \times 62{\cdot}5 \times 15 \times 0{\cdot}5 \times 12{\cdot}5$

$= 211$ lb.-ft. per foot run of wall (tension on the inside of the corner).

The fixing moment along edge of roof slab, assuming the wall is absolutely rigid and the roof slab is relatively flexible,

$= \frac{1}{8} \times 160 \times 12{\cdot}5^2 = 3{,}130$ lb.-ft. per foot run (tension on the outside of the corner).

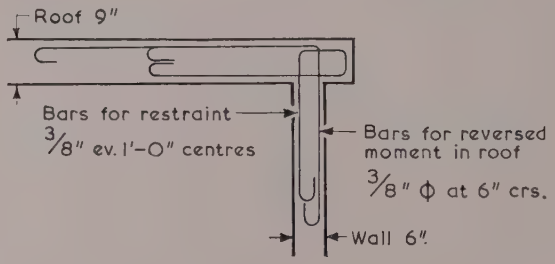


Fig. 4.9.

Even if the thermal expansion of the roof slab is neglected there is a greater likelihood of tension occurring along the outside of the joint than along the inside. A practical designer would supply reinforcement on both sides; a reasonable reinforcement detail for this joint is shown in *Fig.* 4.9.

CHAPTER FIVE

RESTRAINT IN OPEN-TOPPED CIRCULAR TANKS WITH TAPERING WALLS

COMPARING this case of restraint in an open-topped circular tank with a tapering wall, with the simpler case of a wall of uniform thickness it will be seen that the fundamental equation

$$\frac{d^4y}{dh^4} = \frac{wh}{EI} - \frac{yT}{R^2I}$$

still holds good, where h is the depth below water surface and y is the horizontal movement of the wall from its unloaded position. But, whereas with the wall of uniform thickness both T and I are constants, with a tapering wall they both vary (as some function of h). This means that the calculation of any one given case is beyond normal drawing-office practice.

Happily there are available standard results for both limits of this problem—namely, the wall of uniform thickness and the wall that tapers to a knife-edge, and by using these and an approximate method of calculation for other rates of taper results can be interpolated with a greater degree of accuracy than the primary assumptions on which the analysis is based (see *Figs.* 5.3 to 5.8).

Cases of complete restraint are therefore covered by tabulated values, but cases of partial restraint may have to be dealt with by an approximate method such as that which follows.

Approximate Analysis

In *Fig.* 5.1 the wall of a circular tank of radius R and depth H is shown. The thickness of the wall in this example is T at the base and $0{\cdot}5T$ at the top.

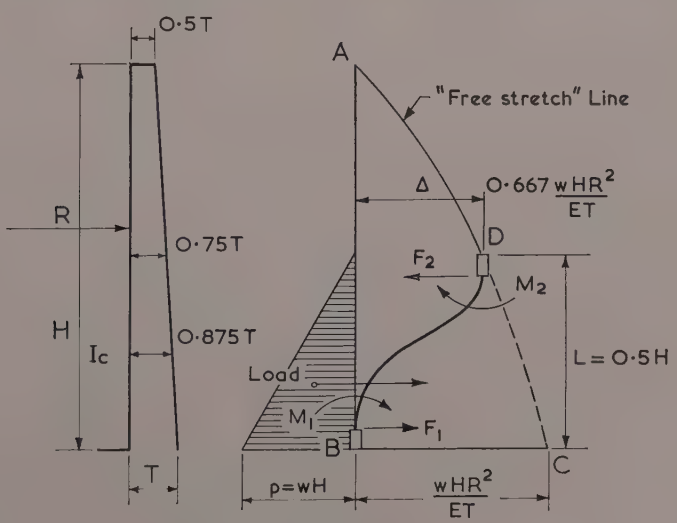


Fig. 5.1.

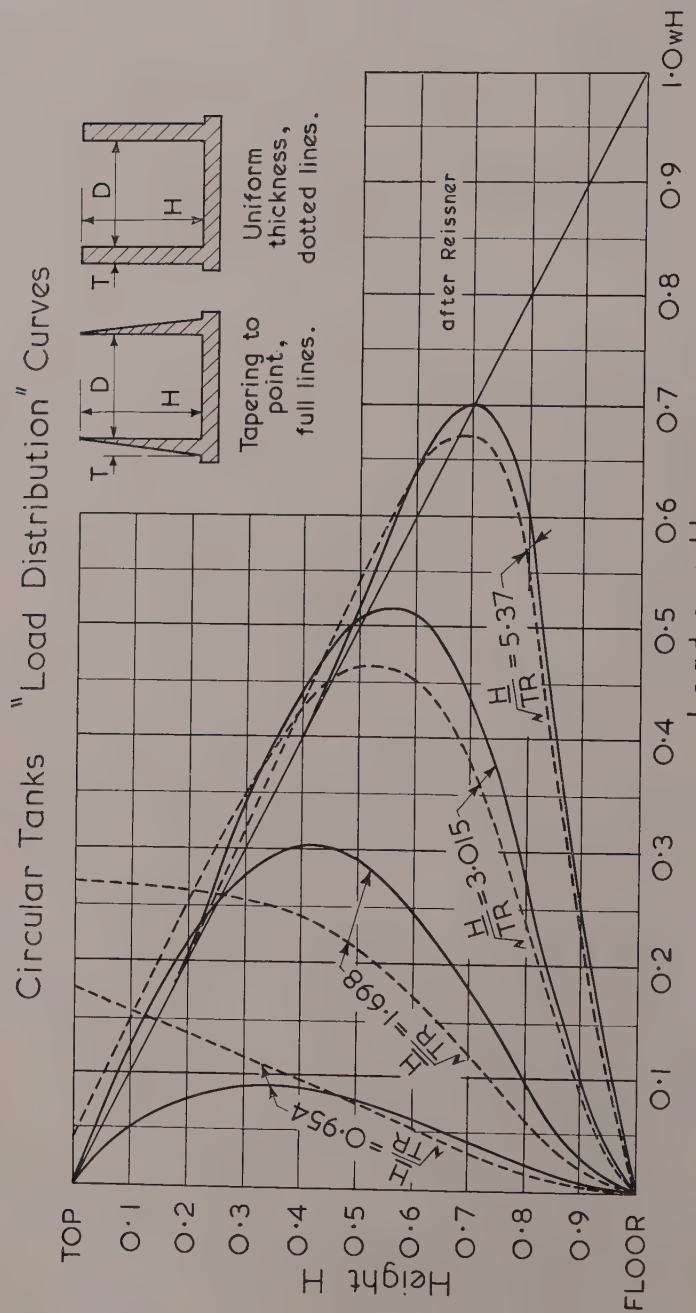

Circular Tanks "Load Distribution" Curves
Tapering to point, full lines.
Uniform thickness, dotted lines.
T
D
H
after Reissner
H/√TR = 5·37
H/√TR = 3·015
√TR/H = 1·698
√TR/H = 0·954
TOP
FLOOR
Height H
Load p = wH
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9
1·0wH

Fig. 5.2.

Draw the vertical line AB representing the original unloaded position of the wall and the 'free-stretch' line ADC showing the position that the wall would take up if it had a frictionless sliding joint at the base. Since the wall varies in thickness, the 'free-stretch' line is no longer a straight line. It is known that, in general, the upper part of the wall will tend to follow (or follow close to) the 'free-stretch' line down to some point D, from which point it will curve back to point B where the base of the wall is fixed in position and direction by the floor. The point D is approximately the point of maximum hoop tension. Since a series of values is required the analysis is commenced by fixing point D at a height of $0{\cdot}5H$. The effective pressure on the vertical 'staves' of the wall is zero at point D where the wall crosses the 'free-stretch' line and has a maximum value of wH at point B. The following approximate assumptions are made:

1. The effective loading on portion BD is triangular.
2. The inclination to the vertical of the tangent slope at point D is zero.
3. The shear force F_2 at D is zero.

In other words D has been displaced horizontally a distance Δ, but there is no rotation of the 'joint' D.

The portion BD consists of a beam $0{\cdot}5H$ long tapering from T to $0{\cdot}75T$. The thickness at midspan is $0{\cdot}875T$ and $I_C = \frac{1}{12} \times (0{\cdot}875T)^3 = 0{\cdot}056T^3$.

Ratio of taper $= T/0{\cdot}75T = 1{\cdot}333$.

Fixed-ended values of M_1 and M_2 (from tables) are

$$M_1 = +0{\cdot}057pL^2 \quad \text{and} \quad M_2 = -0{\cdot}0275pL^2,$$

from which the fixed-ended values of the shearing forces are

$$F_1 = +0{\cdot}3628pL \quad \text{and} \quad F_2 = -0{\cdot}1372pL.$$

The elastic coefficients (from tables) are

$$C_{1B} = 4{\cdot}85;\ C_{1D} = 3{\cdot}18;\ C_{2B} = C_{2D} = 1{\cdot}95.$$

The tabulation is as follows.

	M_1	F_1	M_2	F_2
All joints fixed	$+0{\cdot}057pL^2$	$+0{\cdot}3628pL$	$-0{\cdot}0275pL^2$	$-0{\cdot}1372pL$
Due to Δ	$6{\cdot}80EI_C\frac{\Delta}{L^2}$	$11{\cdot}93EI_C\frac{\Delta}{L^3}$	$5{\cdot}13EI_C\frac{\Delta}{L^2}$	$11{\cdot}93EI_C\frac{\Delta}{L^3}$

Since $F_2 = 0$, $\quad -0{\cdot}1372pL + 11{\cdot}93EI_C\frac{\Delta}{L^3} = 0 \quad$ or $\quad \Delta = \frac{0{\cdot}1372pL^4}{11{\cdot}93EI_C}$.

$$M_1 = +0{\cdot}057pL^2 + 6{\cdot}80EI_C\frac{\Delta}{L^2}.$$

Substituting the value for Δ, $M_1 = +0{\cdot}1351pL^2$.

Now Δ is the free stretch at $D = 0{\cdot}667\frac{wHR^2}{ET}$,

or $$\frac{0{\cdot}1372pL^4}{11{\cdot}93EI_C} = 0{\cdot}667\frac{wHR^2}{ET}.$$

Putting $p = wH$, $L = 0{\cdot}5H$ and $L^4 = 0{\cdot}0625H^4$,

$$\frac{H^4}{T^2R^2} = \frac{0{\cdot}667 \times 11{\cdot}93 \times 0{\cdot}056}{0{\cdot}1372 \times 0{\cdot}0625} = 51{\cdot}9.$$

$$\frac{H^2}{TR} = 7{\cdot}2 \text{ or } H^2 = 7{\cdot}2TR. \quad \frac{H}{\sqrt{TR}} = 2{\cdot}68.$$

$$M_1 \text{ (from foregoing)} = +0{\cdot}1351pL^2 = +0{\cdot}1351wH(0{\cdot}5H)^2$$
$$= +0{\cdot}1351wH \times 0{\cdot}25 \times 7{\cdot}2TR = 0{\cdot}243wHTR.$$

$$\text{Reversed moment } M_2 = -0{\cdot}0275pL^2 + 5{\cdot}13EI_C\frac{\Delta}{L^2} = +0{\cdot}0563wHTR.$$

$$F_1 = 0{\cdot}3628pL + 11{\cdot}93EI_C\frac{\Delta}{L^3} = 0{\cdot}25wH^2.$$

Thus, for a wall that tapers from T to $0{\cdot}5T$ and has a ratio of $H/\sqrt{TR}$ of 2·68, the following approximate values are obtained:

Restraint moment $M_B = 0{\cdot}243wHTR$ (0·21).
Reversed moment $= -0{\cdot}0563wHTR$ (−0·05).
Restraint shear $= 0{\cdot}25wH^2$ (0·25).
Maximum hoop tension $= 0{\cdot}5wHR$ (0·43).
Position of maximum hoop tension $= 0{\cdot}5H$ (0·48) up.

The figures in brackets are more exact values taken from *Figs.* 5.3 to 5.8. It will be seen that the approximate values are within 20 per cent. of the more exact values.

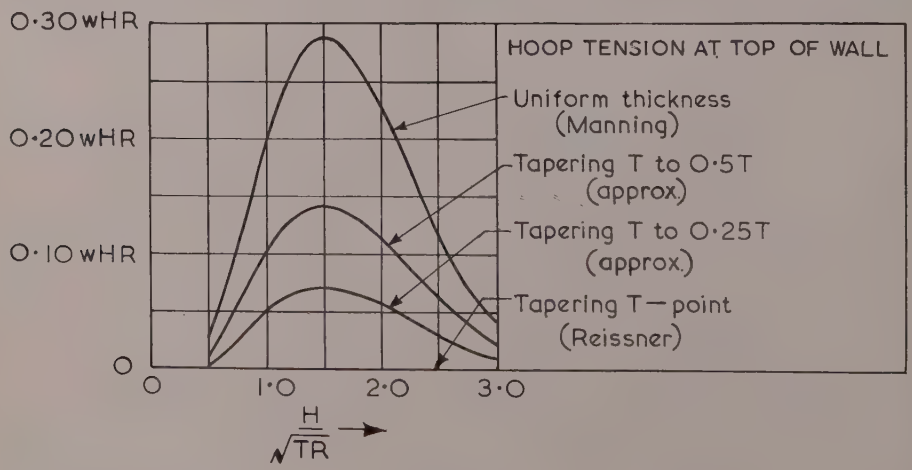


Fig. 5.3.

By fixing point D in *Fig.* 5.1 successively at $0{\cdot}25H$, $0{\cdot}375H$, $0{\cdot}75H$, etc., a series of approximate values may be obtained for a wall that tapers from T to $0{\cdot}5T$. If the procedure is repeated for walls tapering from T to $0{\cdot}33T$, from T to $0{\cdot}25T$ and from T to a knife-edge, and a wall of uniform thickness is included, a complete range of series of approximate values is obtained. But values for the uniform wall and the wall that tapers to a knife-edge have already been calculated (Reissner) and all the results for these other tapered walls must lie between these two limits.

In *Fig.* 5.2 (after Reissner) some typical loading curves are given, showing the hoop tension in uniform and tapered walls for four values of $H/\sqrt{TR}$. From this

figure it will be seen that the only difficulty that may arise is at the top parts of walls with values of $H/\sqrt{TR}$ of less than 3, but here again the two limiting values are known and it is possible to interpolate. The results of such interpolation are given in *Fig.* 5.3. Values of the restraint moment M_B are given in *Fig.* 5.4. Values

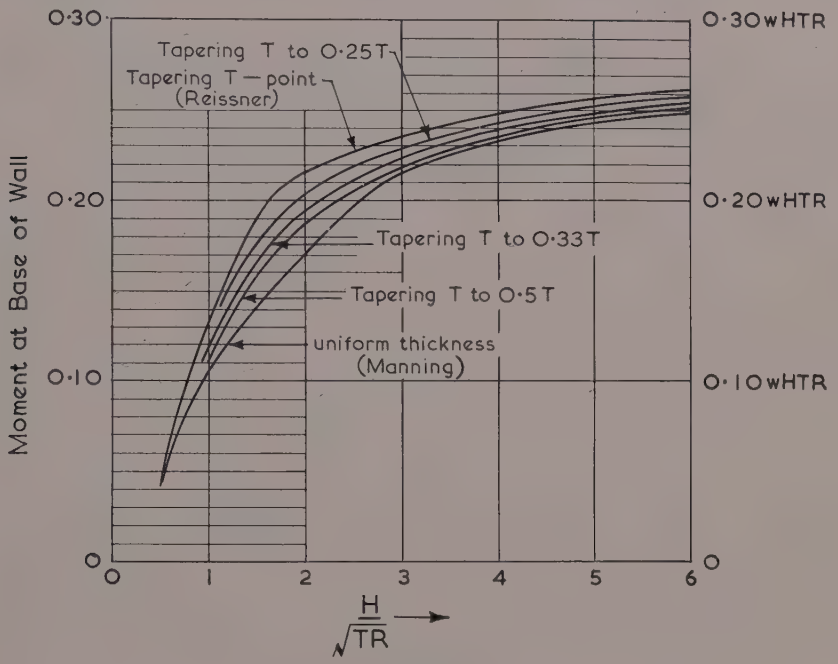


Fig. 5.4.

of the restraint shear are given in *Fig.* 5.5, and the reversed moment in *Fig.* 5.6. Values of the maximum hoop tension are given in *Fig.* 5.7, and the position of the maximum hoop tension in *Fig.* 5.8. By using *Figs.* 5.3, 5.7 and 5.8 a fairly good sketch may be made of the distribution of hoop tension in the wall. As a check this may be compared with *Fig.* 5.2.

To illustrate the use of these curves, consider a water tank 100 ft. in diameter and 25 ft. deep designed to permissible tensile stresses of 200 and 300 lb. per sq. in. in the concrete in direct tension and bending, respectively, and 12,000 lb. per sq. in. in the steel. The wall is to be tapered, the top of the wall being 6 in. thick.

The thickness at the base of the wall will be nearly the same as that for a wall of uniform thickness. Using the graph for a uniform wall (*Fig.* 3.21 in Chapter 3),

$$\frac{\sqrt{H}}{R} = \frac{\sqrt{25}}{50} = 0{\cdot}10.$$

For $100bd^2$, $T = 0{\cdot}1H^2/R = 0{\cdot}1 \times 25^2/50 = 1{\cdot}25$ ft.
For $50\ bd^2$, $T = 0{\cdot}17H^2/R = 2{\cdot}12$ ft.

To begin, alternative designs will be prepared, (a) with $T = 1{\cdot}5$ ft. and (b) with $T = 2{\cdot}0$ ft.

With design (a), $\dfrac{H}{\sqrt{TR}} = \dfrac{25}{\sqrt{1{\cdot}5 \times 50}} = 2{\cdot}89.$

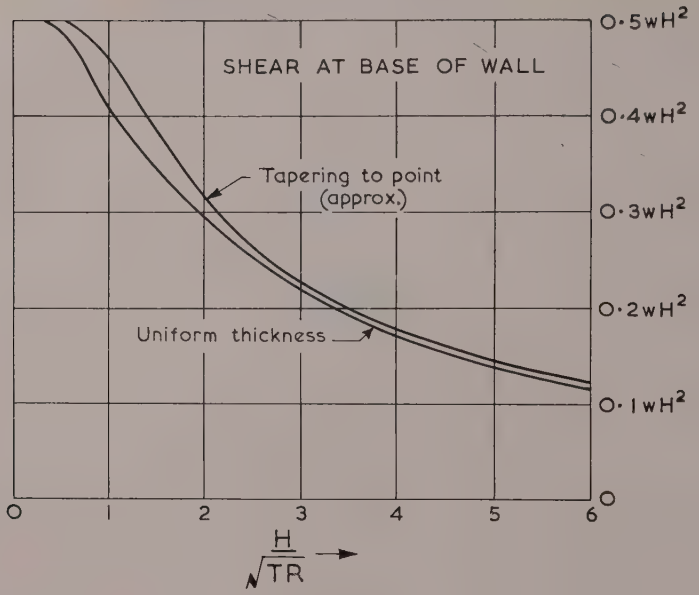


Fig. 5.5.

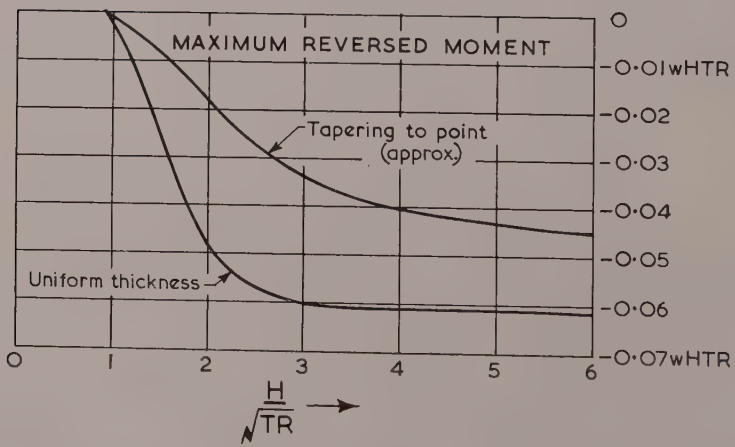


Fig. 5.6.

Taper = 1·5 ft. to 0·5 ft. = T to 0·33T.

From *Fig.* 5.4, $M_B = 0{\cdot}222wHTR$. From *Fig.* 5.7, maximum hoop tension $= 0{\cdot}48wHR$. From *Fig.* 5.8, position of maximum tension is $0{\cdot}45H$ up. From *Fig.* 3.3, top tension $= 0{\cdot}02wHR$. From *Fig.* 3.5, shear at base $= 0{\cdot}23wH^2$. From *Fig.* 3.6, reversed moment $= 0{\cdot}045wHTR$, say.

$$M_B = 0{\cdot}222 \times 62{\cdot}5 \times 25 \times 1{\cdot}5 \times 50$$
$$= 26{,}000 \text{ lb.-ft. per foot run of wall.}$$
$$= 312{,}000 \text{ lb.-in.}$$

$$\frac{M}{bd^2} = \frac{312{,}000}{12 \times 18^2} = 80{\cdot}3.$$

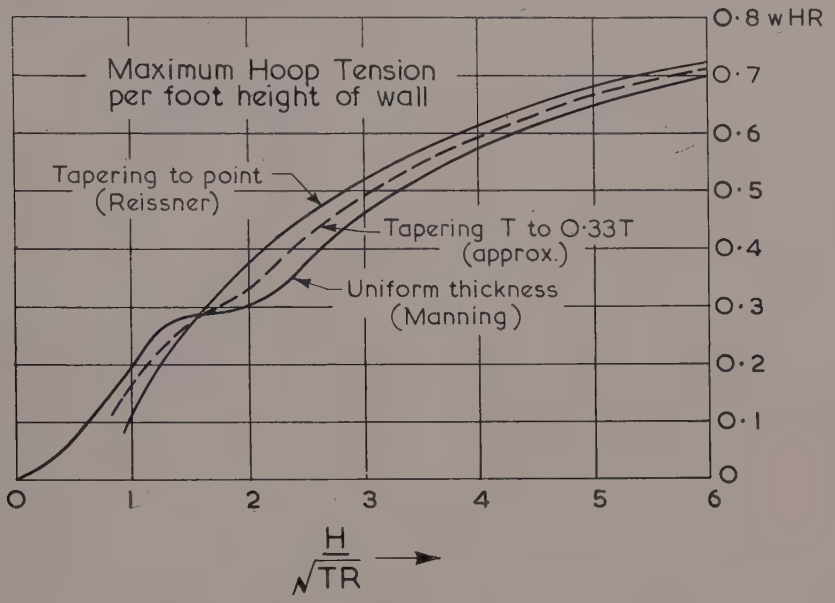


Fig. 5.7.

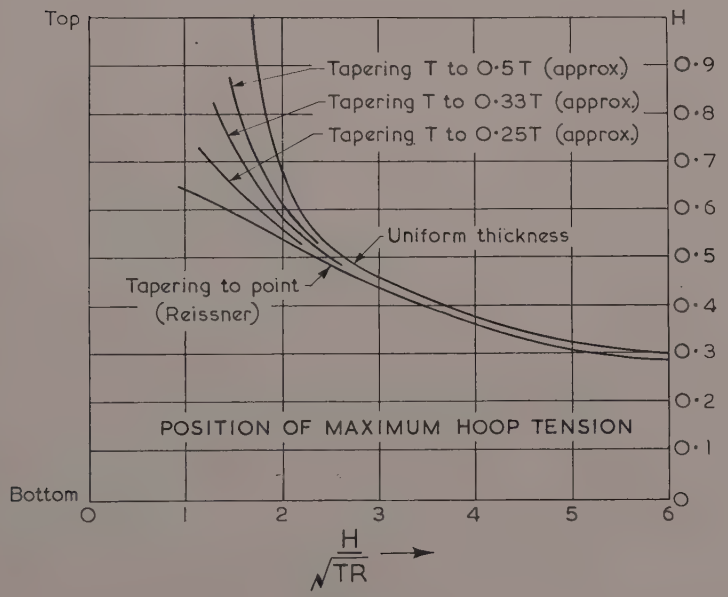


Fig. 5.8.

To resist this 1·6 per cent. of tension steel alone, or 1·2 per cent. of steel on both sides, is required. That is 3·45 sq. in. of steel near the inside face of the wall or 2·59 sq. in. of steel near both faces.

The maximum hoop tension is $0{\cdot}48 \times 62{\cdot}5 \times 25 \times 50 = 37{,}500$ lb.

This occurs $0{\cdot}45H$ above the floor of the tank. At this point the wall is 12·6 in. in thickness.

$$\text{Hoop steel required} = \frac{37{,}500}{12{,}000} = 3{\cdot}13 \text{ sq. in.}$$

$$\text{Tensile stress on concrete} = \frac{37{,}500}{(12 \times 12{\cdot}6) + (14 \times 3{\cdot}13)} = 193 \text{ lb. per sq. in.}$$

$$\text{With design (b), } \frac{H}{\sqrt{TR}} = \frac{25}{\sqrt{2 \times 50}} = 2{\cdot}5.$$

Taper = 2·0 ft. to 0·5 ft. = T to $0{\cdot}25T$.
From *Fig.* 5.4, $M_B = 0{\cdot}22wHTR$. From *Fig.* 5.7, maximum hoop tension $= 0{\cdot}45wHR$. From *Fig.* 5.8, position of maximum tension is $0{\cdot}5H$ up. From *Fig.* 5.3, top tension $= 0{\cdot}03wHR$. From *Fig.* 5.5, shear at base $= 0{\cdot}26wH^2$. From *Fig.* 5.6, reversed moment $= 0{\cdot}035\ wHTR$, say.

$$M_B = 0{\cdot}22 \times 62{\cdot}5 \times 25 \times 2 \times 50$$
$$= 34{,}400 \text{ lb.-ft. per foot run of wall}$$
$$= 413{,}000 \text{ lb.-in.}$$
$$\frac{M}{bd^2} = \frac{413{,}000}{12 \times 24^2} = 59{\cdot}7.$$

To resist this, 0·65 per cent. of tension steel will suffice or 1·87 sq. in. of steel near the water face. The maximum hoop tension is

$$0{\cdot}45 \times 62{\cdot}5 \times 25 \times 50 = 35{,}100 \text{ lb.}$$

This tension occurs at $0{\cdot}5H$ above the floor, where the wall is 15 in. in thickness.

$$\text{Hoop steel required} = \frac{35{,}100}{12{,}000} = 2{\cdot}93 \text{ sq. in.}$$

$$\text{Tensile stress in concrete} = \frac{35{,}100}{(12 \times 15) + (14 \times 2{\cdot}93)} = 159 \text{ lb. per sq. in.}$$

Before it can be decided which of these two designs is more suitable, it is necessary to sketch out the wall and the edge of the floor and make preliminary estimates of the amount of steel and concrete required for each design. Sketches of both designs, with the hoop tensions sketched in, are shown in *Fig.* 5.9.

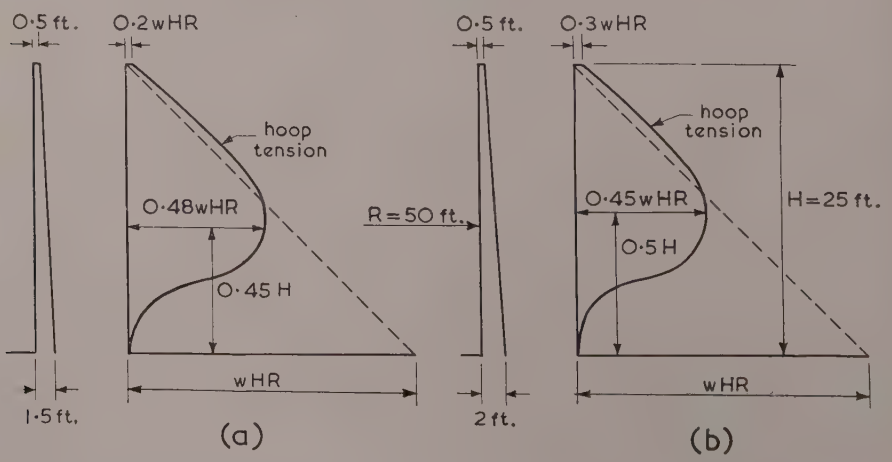


Fig. 5.9.

Partial Restraint

In cases where partial restraint occurs, the approximate method as shown in *Fig.* 5.1 may be used to obtain an approximate result but this entails several trials for any given value of $H/\sqrt{TR}$. If the floor in *Fig.* 5.1 stretches and point B moves outward an amount $0{\cdot}25\frac{wHR^2}{ET}$, then $p = 0{\cdot}75wH$ and $\Delta = (0{\cdot}667 - 0{\cdot}25)\frac{wHR^2}{ET}$, giving values of $H/\sqrt{TR} = 2{\cdot}56$ and $M_B = 0{\cdot}167wHTR$.

By moving point D up and down, a series of corresponding values of $H/\sqrt{TR}$, L and M_B are obtained. By drawing these as curves, any required value of $H/\sqrt{TR}$ can be selected and the corresponding values of L and M_B read off.

If the actual value of $H/\sqrt{TR}$ in the example considered is more than 2·56, point D (the point of maximum hoop tension) must be moved downwards; if less,

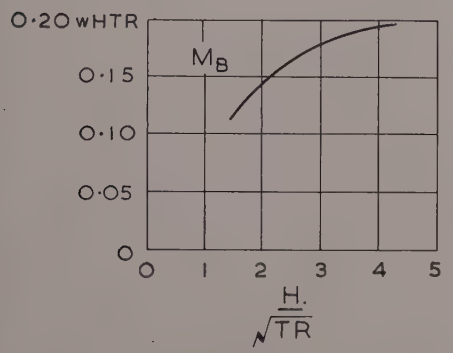


Fig. 5.10.

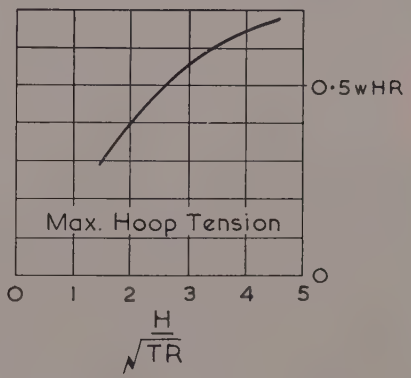


Fig. 5.11.

then point D must be moved upwards. If point D is actually moved up to $0{\cdot}667H$ (still allowing for the outward movement of $0{\cdot}25wHR^2/ET$ at point B), $H/\sqrt{TR} = 1{\cdot}638$ and $M_B = 0{\cdot}125wHTR$. If point D is moved down to $0{\cdot}333H$, then $H/\sqrt{TR} = 4{\cdot}22$ and $M_B = 0{\cdot}1955wHTR$. Now M_B may be plotted against $H\sqrt{TR}$ (*Fig.* 5.10) and also the maximum hoop tension may be plotted against $H/\sqrt{TR}$ (*Fig.* 5.11). If the actual wall being considered has a value of $H/\sqrt{TR}$ of 3 than $M_B = 0{\cdot}18wHTR$ (*Fig.* 5.10) and the maximum hoop tension is $0{\cdot}55wHR$ (*Fig.* 5.11); that is, point D is $0{\cdot}45H$ above the floor.

These results may be checked by calculating the moments and tensions in a wall of uniform thickness with the same value of $H/\sqrt{TR}$. It must be remembered that *Figs.* 5.10 and 5.11 apply only to the particular wall in *Fig.* 5.1 under the conditions of 75 per cent. restraint. The reader must draw pairs of similar curves for every case he wishes to analyse.

CHAPTER SIX

RESTRAINT IN LARGE-DIAMETER SHALLOW CIRCULAR TANKS

If a tank is only 20 ft. in diameter and 40 ft. deep it should clearly be designed as a circular tank with the wall in hoop tension, proper provision being made for restraint moments and shear (or a sliding bottom joint being provided). If a tank is 200 ft. in diameter and only 6 ft. deep the wall may be designed as a simple cantilever. Practice confirms that, with a tank of these dimensions, such a design is satisfactory. The question therefore arises—at what point is it possible or desirable to change from a hoop tension design to a simple cantilever design? There are really two problems here:

1. At what stage is it structurally possible to change.
2. At what stage is it economical (or more convenient) to change.

If the tank is to have an elastic waterproof lining and it does not matter how badly the reinforced concrete cracks, a solution for the first of these two problems is unnecessary. But if an uncracked unlined tank is to be provided then the outward deflection of the tank wall must be considered and the effects of slight movements of the foundations and of the tank floor must be taken into account.

With a shallow tank of large diameter it may be cheaper to resist all the water pressure by cantilever action, but the top ends of the cantilevers move outwards and this may cause cracks to form, running down from the top of the wall and giving rise to unsightly leakage. (Slurry tanks designed in this way have actually cracked and leaked badly.) In some cases this trouble could be avoided by cutting the wall into a series of short arcs by the introduction of expansion joints (with metal or plastic water-bars), but this practice may lead to trouble if the top of the wall has to carry a continuous rail supporting some type of stirring or clarifying apparatus.

The amount of outward movement that may be tolerated before appreciable cracking occurs can be arrived at by considering the design of circular walls in hoop tension. It is known that if the calculated hoop tension in the concrete is restricted to about 200 lb. per sq. in., the wall will not crack seriously. Before this tension can develop the wall must stretch by $200/E_c$ (length). This means that the radius must also increase pro-rata by $200/E_c$ (radius). If the radius of the tank is 100 ft. (1,200 in.) and the wall is stressed to 200 lb. per sq. in. there will be an outward movement of the wall of $\frac{200}{E_c} \times 1{,}200$ and if $E_c = 3{,}000{,}000$ lb. per sq. in., this movement is $\frac{200 \times 1{,}200}{3{,}000{,}000} = 0{\cdot}080$ in. Therefore, if the radius of the tank is 100 ft. the wall may safely be designed as a simple cantilever provided that the outward deflection of the top end does not exceed about 0·080 in.

To state the general case—the wall may safely be designed as a simple cantilever provided the outward deflection of the top does not exceed $\frac{f_t}{E_c} \times$ (radius), where f_t is usually taken as 200 to 300 lb. per sq. in.

With a cantilever wall of uniform thickness T as shown in *Fig.* 6.1 loaded with a triangular load W per unit length, the deflection of the top end is:

$$\Delta = \frac{WH^3}{15E_cI} = \frac{wH^5}{30E_cI}.$$

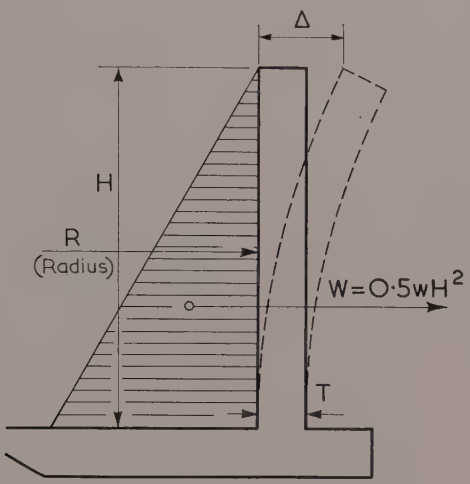


Fig. 6.1.

Writing $I = \frac{1}{12}T^3$, then $\Delta = \dfrac{wH^5 \times 12}{30E_cT^3}$. If the deflection is limited to f_tR/E_c, then

$$\frac{wH^5 \times 12}{30E_cT^3} = f_t\frac{R}{E_c} \quad \text{or} \quad T^3 = \frac{2wH^5}{5f_tR}.$$

In this expression all the terms must be expressed in the same units. If T, H and R are in feet, then w must be in lb. per cu. ft. and f_t must be in lb. per sq. ft. The cantilever moment at the base of the wall is $\frac{1}{6}wH^3$. If the bottom section of the wall is designed with an allowable stress f_b in bending (usually about 300 lb. per sq. in.), then

$$\tfrac{1}{6}wH^3 = \tfrac{1}{6}f_bT^2 \quad \text{or} \quad T^2 = wH^3/f_b.$$

If the top end of the wall has reached its maximum allowable extension, then (as already shown) $T^3 = \dfrac{2wH^5}{5f_tR}$. Dividing one expression by the other,

$$T = \frac{2H^2f_b}{5f_tR} \quad \text{or} \quad \frac{H}{\sqrt{TR}} = 1{\cdot}58\sqrt{\frac{f_t}{f_b}}.$$

If $f_t = 200$ lb. per sq. in. in direct tension and $f_b = 300$ lb. per sq. in. in bending tension, $H/\sqrt{TR} = 1{\cdot}29$ for a wall of uniform thickness. The expression has been put in this form so that it may be compared with deep narrow circular tanks where $H/\sqrt{TR}$ is the critical value.

If the wall tapers towards the top the deflection is increased and the limiting value of $H\sqrt{TR}$ decreases, as shown in *Fig.* 6.2.

Reverting to the wall of uniform thickness two expressions have been obtained for T.

If the first is cubed and the second is squared,

$$T^6 = \frac{w^3H^9}{f_b^3} = \frac{4w^2H^{10}}{25f_t^2R^2} \quad \text{or} \quad R^2 = \frac{4f_b^3}{25wf_t^2}H.$$

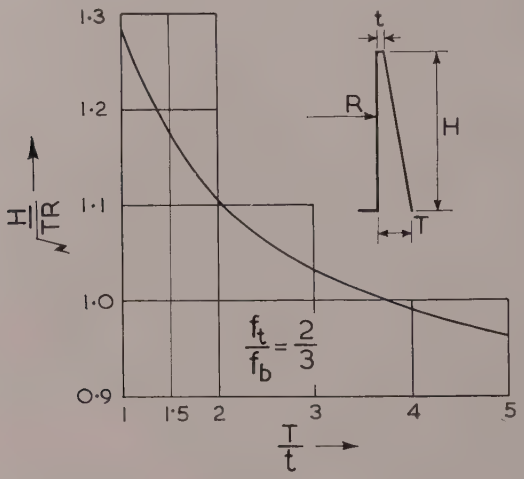


Fig. 6.2.

If $w = 62{\cdot}5$ lb. per cu. ft., $f_b = 300 \times 144$ lb. per sq. ft. and $f_t = 200 \times 144$ lb. per sq. ft., then $R = 15{\cdot}78\sqrt{H}$.

If $H = 25$ ft, then $R = 15{\cdot}78 \times 5 = 78{\cdot}9$ ft.

Taking other values of H and modifying the coefficients to allow for tapering walls, the values given in *Fig.* 6.3 are obtained. This figure is not put forward as an exact solution to the problem, as the reader may wish to work to other stresses and will usually want to make some allowance for the area of steel reinforcement when finally checking his design. The values in *Fig.* 6.3 are only correct if f_t is exactly 300 lb. per sq. in. and f_t is exactly 200 lb. per sq. in. when reckoned on the concrete section alone. An actual example should make the matter clearer.

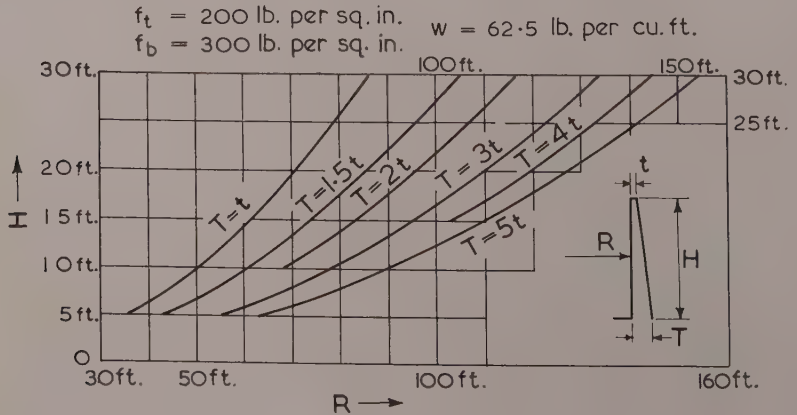


Fig. 6.3.

A circular water tank is 15 ft. deep and 120 ft. in diameter. If this is designed for maximum tensile stresses of 200 lb. per sq. in. (in direct tension) and 300 lb. per sq. in. (in bending), can the wall be designed as a cantilever?
Cantilever $M = \frac{1}{6} \times 62{\cdot}5 \times 15^3 \times 12 = 422{,}000$ lb.-in.

$$\text{Minimum } d = \sqrt{\frac{422{,}000}{50 \times 12}} = 26{\cdot}5 \text{ in., say } 27 \text{ in., or } T = 2{\cdot}25 \text{ ft.}$$

Making the wall 9 in. thick at the top and tapering uniformly from bottom to top, $t = 0{\cdot}75$ ft. (see *Fig.* 6.5).

$$\frac{T}{t} = \frac{2{\cdot}25}{0{\cdot}75} = 3{\cdot}0.$$

The thickness half-way up is 18 in. = 1·5 ft.
I_c per unit length of wall $= \frac{1}{12} \times 1{\cdot}5^3 = 0{\cdot}281$ ft.^{4} From *Fig.* 6.4,

$$\Delta = 0{\cdot}0308 \frac{WH^3}{E_c I_c} = \frac{0{\cdot}0308 \times (0{\cdot}5 \times 62{\cdot}5 \times 15^2) \times 15^3}{E_c \times 0{\cdot}281} = \frac{2{,}592{,}000}{E_c} \text{ ft.,}$$

where E_c is expressed in lb. per sq. ft.

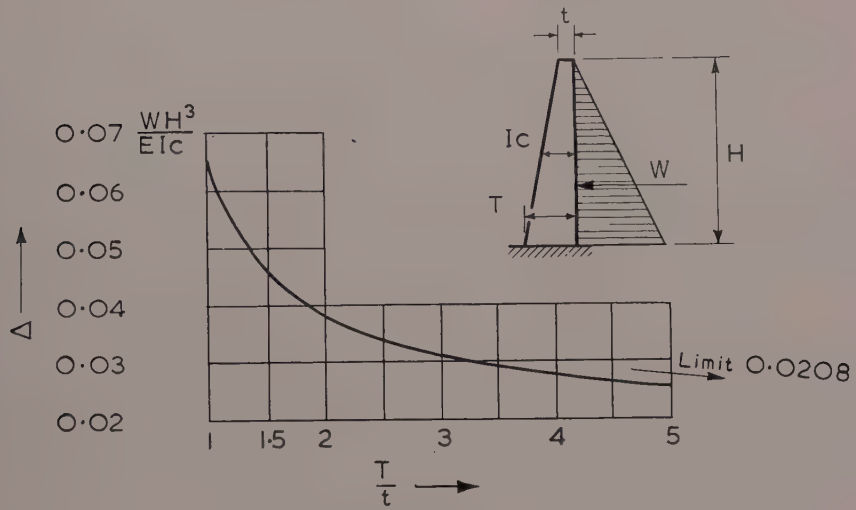


Fig. 6.4.

The maximum allowable extension of the radius is

$$\frac{f_t}{E_c} R = \frac{(200 \times 144) \times 60}{E_c} = \frac{1{,}730{,}000}{E_c} \text{ ft.}$$

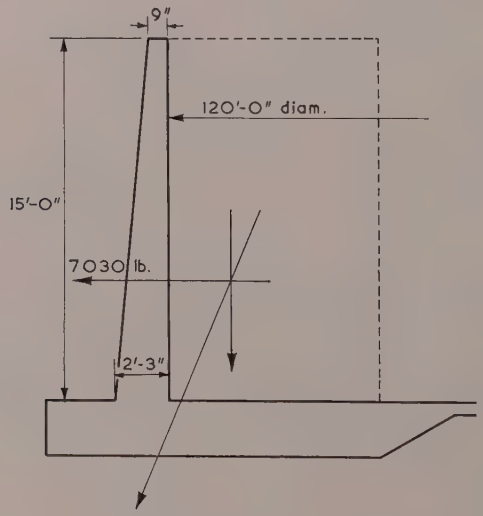
whereas $\Delta = \frac{2{,}592{,}000}{E_c}$ ft., or 50 per cent. greater than the maximum allowable strain $f_t R/E_c$. The circumferential strain at the extreme top of the wall is therefore the same as would be caused by a hoop tension of 300 lb. per sq. in. Most practical designers, if they had the choice, would accept this, since the maximum strain occurs only at the very tip of the wall and some horizontal steel must be provided in the wall (according to CP.2007:1960, 0·3 per cent. of plain bars or 0·25 per cent. of deformed bars). If the horizontal bars are hooked at both ends and given ample laps, they must take some tension near the top of the wall, thus relieving some of

the cantilever load. If the concrete strain corresponds to a concrete stress of 300 lb. per sq. in. and m is equal to 15, the steel is stressed to 300 × 15 or 4,500 lb. per sq. in.

If no deviation whatever is allowed from the value of 200 lb. per sq. in., then Δ may be reduced by increasing I_c by 50 per cent., which means increasing the thickness by about 15 per cent. If the thickness at the base of the wall were increased to 33 in., this would give a thickness halfway up the wall of 21 in. The value of I_c would then be $\frac{1}{12} \times 1{\cdot}75^3$ or 0·447 ft.^{4} and

$$\frac{T}{t} = \frac{33}{9} = 3{\cdot}67.$$

The reader may check this for himself. Note that a wall tapering from 27 in. to 9 in. would be quite satisfactory for a radius of 1·5 × 60 or 90 ft. This could have been seen at once from *Fig.* 6.3. For a depth of 15 ft. and a taper of $T = 3t$ the minimum radius is 95 ft. The discrepancy between 90 ft. and 95 ft. is due to the fact that the thickness of the wall was made 27 in. instead of 26·5 in. An increase of 1·89 per cent. in thickness reduces the deflection by 5·8 per cent. The wall is shown in *Figs.* 6.5 and 6.7.



CANTILEVER

Fig. 6.5.

To see whether a cantilever design is economical it may be compared with a design based on hoop tension. An approximate thickness for T may be obtained from the curve for a uniform restrained wall (*Fig.* 3.21).

$\frac{\sqrt{H}}{R} = \frac{\sqrt{15}}{60} = 0{\cdot}0645$. At $50bd^2$, $T = 0{\cdot}34\frac{H^2}{R} = \frac{0{\cdot}34 \times 15^2}{60} = 1{\cdot}275$ ft., say 16 in. If the top of the wall is made 8 in. in thickness, then $T/t = 16/8 = 2$. The wall is shown in *Fig.* 6.6.

$$\frac{H}{\sqrt{TR}} = \frac{15}{\sqrt{1{\cdot}33 \times 60}} = 1{\cdot}68.$$

From the curves in *Figs.* 5.3 to *Fig.* 5.8,

Restraint $M_B = 0{\cdot}17wHTR = 0{\cdot}17 \times 62{\cdot}5 \times 15 \times 1{\cdot}33 \times 60 = 12{,}750$ lb.-ft.
Restraint shear $= 0{\cdot}34wH^2 = 0{\cdot}34 \times 62{\cdot}5 \times 15^2 = 4{,}780$ lb.
Maximum hoop tension $= 0{\cdot}3wHR = 16{,}900$ lb.
Position of maximum tension is $0{\cdot}72H$ above floor of tank.
Hoop tension at top of wall $= 0{\cdot}17wHR = 9{,}600$ lb.
Reversed moment say $0{\cdot}02wHTR = 1{,}500$ lb.-ft.

$M_B = 12{,}750$ lb.-ft. $= 153{,}000$ lb.-in. per foot run of wall.

$$\frac{M}{bd^2} = \frac{153{,}000}{12 \times 16^2} = 49{\cdot}8.$$

$$A_{st} = \frac{153{,}000}{12{,}000 \times 0{\cdot}9 \times 14} = 1{\cdot}01 \text{ sq. in., say } \tfrac{3}{4}\text{-in. diam. bars at 5-in. centres.}$$

$$\text{Hoop steel} = \frac{16{,}900}{12{,}000} = 1{\cdot}41 \text{ sq. in.}$$

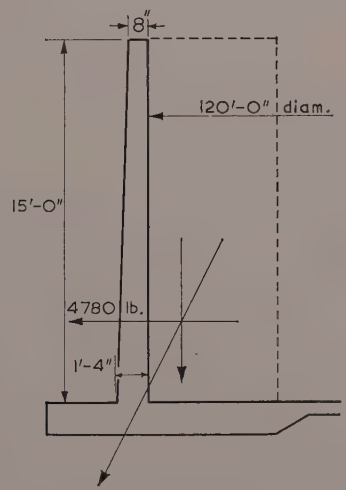
Near outside face: $\frac{3}{4}$-in. diam. bars at $5\frac{1}{2}$-in. centres $= 0{\cdot}96$ sq. in.
Near inside face: $\frac{3}{4}$-in. diam. bars at 11-in. centres $= 0{\cdot}48$ sq. in.
$1{\cdot}44$ sq. in.

Maximum hoop tension occurs where wall is 10·25 in. in thickness.

$$\text{Tensile stress in concrete alone} = \frac{16{,}900}{12 \times 10{\cdot}25} = 138 \text{ lb. per sq. in.}$$

The curve of hoop-tension loading and a section through the wall are shown in *Figs.* 6.6 and 6.8.

Before a comparison of costs can be made a knowledge of the site conditions is



HOOP TENSION
Fig. 6.6.

required as the design of the base of the wall depends on these conditions. Considering the walls and bases to be as shown in *Figs.* 6.5, 6.6, 6.7 and 6.8, the cantilevered wall requires 2·0 cu. yd. of concrete and 2·5 cwt. of reinforcement per foot run of wall while the hoop-tension wall requires only 1·1 cu. yd. of concrete and

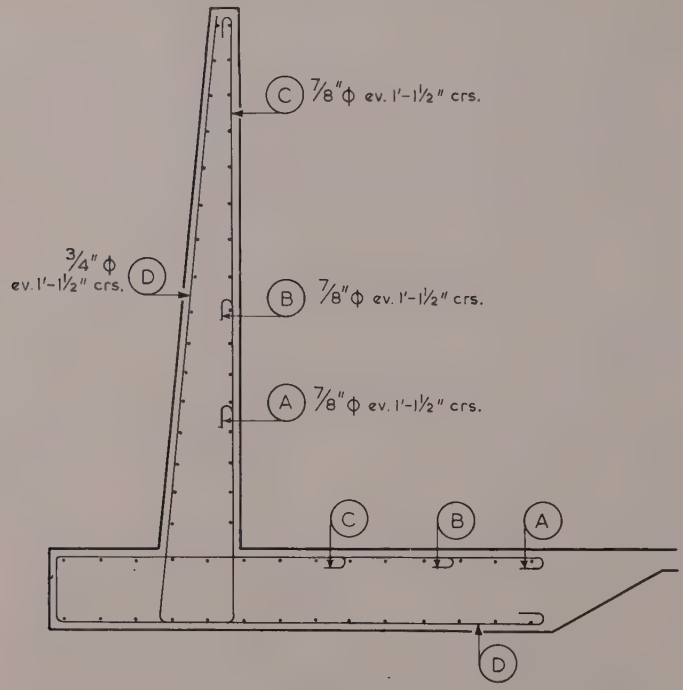
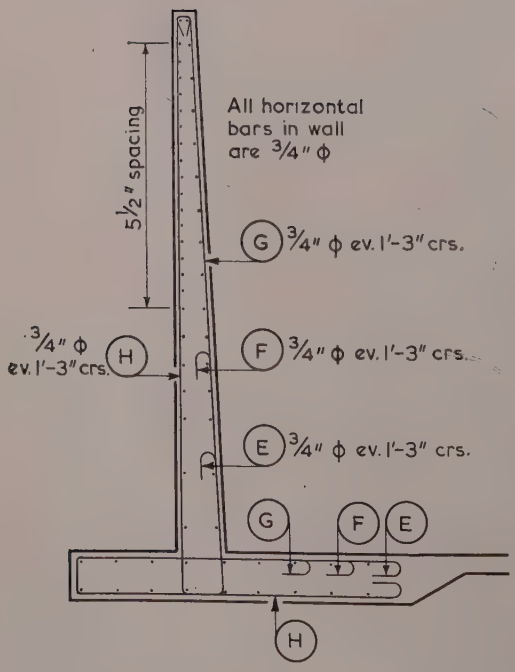

C 7/8" ϕ ev. 1'-1½" crs.
3/4" ϕ ev. 1'-1½" crs. D
B 7/8" ϕ ev. 1'-1½" crs.
A 7/8" ϕ ev. 1'-1½" crs.
C B A
D

Fig. 6.7.


5½" spacing
All horizontal bars in wall are 3/4" ϕ
G 3/4" ϕ ev. 1'-3" crs.
3/4" ϕ ev. 1'-3" crs. H
F 3/4" ϕ ev. 1'-3" crs.
E 3/4" ϕ ev. 1'-3" crs.
G F E
H

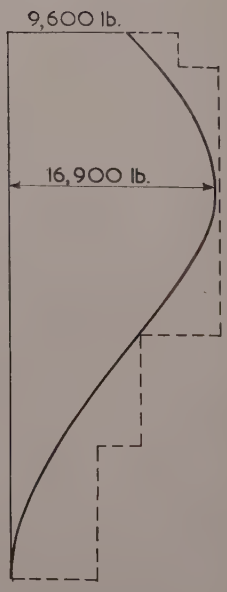

9,600 lb.
16,900 lb.

Fig. 6.8.

1·8 cwt. of reinforcement. For a tank having a diameter of 120 ft. and a depth of 15 ft. the cantilever design is clearly much dearer. One of the reasons for the difference is that the cantilever wall must contain a minimum of 0·3 per cent. of horizontal distribution steel which takes no part in resisting the water pressure. In the hooped wall both vertical and horizontal bars are structurally effective.

In *Fig.* 6.9 alternative cantilever and hoop-tension designs for the wall of a tank

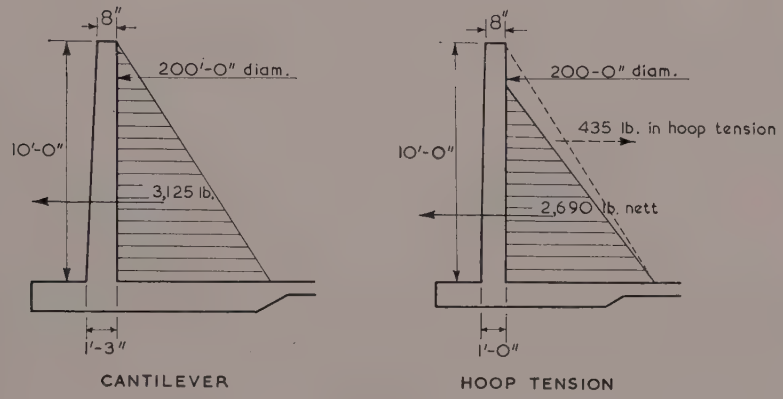


Fig. 6.9.

200 ft. in diameter and 10 ft. deep are shown. With these dimensions and using a wall 12 in. in thickness at the base, with the hoop-tension design 86 per cent. of the water pressure is carried by the restraint shear and only 14 per cent. by hoop tension. Compare this with the wall in *Fig.* 6.8 where 68 per cent. is carried by the restraint shear and 32 per cent. by hoop tension.

The quantities for the wall and footing (as shown in *Fig.* 6.9) per foot run of wall are:

Cantilever design: 0·795 cu. yd. of concrete and 1·09 cwt. of steel.
Hoop-tension design: 0·605 cu. yd. of concrete and 0·94 cwt. of steel.

Even with a depth of 10 ft. and a diameter of 200 ft. the hoop-tension scheme is cheaper. There is no need to check the outward deflection of the top of the cantilever wall; from *Fig.* 6.3, a 10-ft. wall tapering from T to $0{\cdot}5T$ requires a minimum radius of 67 ft. against the actual radius provided of 100 ft.

Movement of Wall Base

If the base of the wall moves outward the circumference of the tank increases pro rata. If the radius is 100 ft. (1,200 in.) and the wall base moves out 0·10 in., the strain in the wall is 0·10/1,200, and the induced stress, assuming E_c is 3,000,000 lb. per sq. in., is $\frac{0{\cdot}10}{1{,}200} \times 3{,}000{,}000 = 250$ lb. per sq. in. This, added to the stresses caused by loading, would almost certainly cause cracks in the upper part of a cantilevered wall.

The same remarks would apply if the base did not move horizontally but settled unevenly. If the wall were 15 ft. high and the base 15 ft. wide, and the outside edge settled 0·10 in. more than the inside edge, the top of the wall would move out about 0·10 in.

With a cantilevered wall, outward movement of the base does not affect the effective water pressure on the wall. With a hooped wall, outward movement reduces the restraint moment and the restraint shear, and increases the hoop tension in the lower part of the wall, thus reducing the effective outward water pressure at the base of the wall. If the wall shown in *Figs.* 6.6 and 6.8 had a sliding joint at the bottom the maximum hoop tension would be $15 \times 62{\cdot}5 \times 60 = 56{,}200$ lb. per foot of height. The wall is 16 in. in thickness and the hoop stress in the bottom of the wall (ignoring the steel) would be $\frac{56{,}200}{16 \times 12} = 293$ lb. per sq. in. This would cause a strain of $\frac{293}{E_c}$ in the wall and an increase of $\frac{293}{E_c} \times 60$ ft. in the radius. Taking E_c as 3,000,000 lb. per sq. in., the increase would be

$$\frac{293}{3{,}000{,}000} \times 60 \times 12 = 0{\cdot}0703 \text{ in.}$$

Thus, if the base of this wall moved outward by 0·0703 in. the restraint moment and restraint shear would vanish and the wall would be in the same state of simple hoop tension as a wall with an effective sliding joint at the bottom, and the amount of hoop steel provided would have to be greatly increased.

Outward movement of the base of the wall may be restricted by reinforcement in the tank floor (provided the floor is not severed by drainage or pipe trenches). The total water pressure on the cantilevered wall in *Figs.* 6.5 and 6.7 is 7,030 lb. per foot run. If all this were taken on a 6-in. floor slab the stress (ignoring the steel) would be $\frac{7{,}030}{6 \times 12} = 98$ lb. per sq. in. and the strain about $\frac{98}{3{,}000{,}000}$, causing the wall to move out about $\frac{98}{3{,}000{,}000} \times 60$ ft., or 0·0235 in. These figures are generally only of academic interest. If the ground is so poor that no frictional resistance can be relied on, the tank floor will be carried on piles, piers or sleeper walls.

Example

In *Fig.* 6.10, the wall of a sedimentation tank 75 ft. in diameter and 9 ft. 6 in. deep is shown. The wall carries a continuous outlet trough, which runs all round the tank as shown. This trough is 3 ft. wide and 1 ft. 3 in. deep, the bottom being screeded to falls. The outer wall of the trough carries a rolling load of 1,120 lb. which is spread over a length of 5 ft., giving 224 lb. per foot. The normal water level is at the top of the main wall, but the tank is to be designed for a head of 9 ft. 9 in. in case the outlets choke. The contents weigh 64 lb. per cu. ft.

$$H = 9{\cdot}75 \text{ ft.} \qquad R = 37{\cdot}5 \text{ ft.}$$

$$\frac{\sqrt{H}}{R} = \frac{3{\cdot}12}{37{\cdot}5} = 0{\cdot}0832. \qquad \frac{H^2}{R} = \frac{9{\cdot}75^2}{37{\cdot}5} = 2{\cdot}54.$$

The approximate thickness may be found from *Fig.* 3.21. Working to $50bd^2$,

$$T = 0{\cdot}22\frac{H^2}{R} = 0{\cdot}22 \times 2{\cdot}54 = 0{\cdot}56 \text{ ft.} = 6{\cdot}7 \text{ in.}$$

With horizontal and vertical bars on both faces and 1½-in. concrete cover, a 7-in.

wall is not going to leave much room down the centre for concreting; consequently an 8 in. wall will be chosen for a preliminary design and the same thickness will be assumed for the floor and outer wall of the trough, and the outer part of the main floor.

If $T = 8$ in. $= 0{\cdot}67$ ft.

$$\frac{H}{\sqrt{TR}} = \frac{9{\cdot}75}{\sqrt{0{\cdot}67 \times 37{\cdot}5}} = 1{\cdot}95.$$

This is more than the critical value of 1·29 and consequently this wall should not

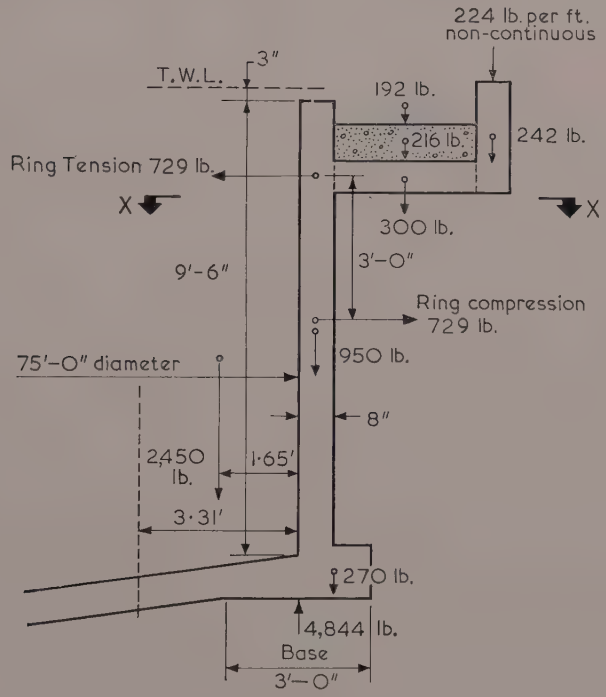


Fig. 6.10.

be designed as a simple cantilever; instead it will be designed as a fully-restrained circular tank. From *Fig.* 3.13, the moment at the base of the wall $= 0{\cdot}175wHTR$

$= 0{\cdot}175 \times 64 \times 9{\cdot}75 \times 0{\cdot}67 \times 37{\cdot}5 = 2{,}740$ lb.-ft. per foot.
$= 32{,}800$ lb.-in. per foot run of wall.

From *Fig.* 3.16, maximum ring tension $= 0{\cdot}29wHR$

$= 0{\cdot}29 \times 64 \times 9{\cdot}75 \times 37{\cdot}5 = 6{,}780$ lb. per foot height.

Since the sectional area of an 8-in. wall per foot is 96 sq. in., the ring tension in the concrete is less than 100 lb. per. sq. in.

$$\text{Maximum ring steel} = \frac{6{,}780}{12{,}000} = 0{\cdot}565 \text{ sq. in.}$$

If $\frac{1}{2}$-in. diam. bars are provided at 6-in. centres in the outside rings (Bars E in *Fig.* 6.11), and $\frac{1}{2}$-in. diam. bars are provided at 12-in. centres in the inside rings (Bars F in *Fig.* 6.11) the total area of steel is 0·589 sq. in. per foot height of wall.

From *Fig.* 3.18, the maximum ring tension occurs at a height of $0{\cdot}7H$ or

6 ft. 10 in. from the floor. The shape of the wall is close to the shape for a value of $H/\sqrt{TR}$ of 2·0 in. *Fig.* 3.4. This indicates how the ring tension falls off towards the bottom of the wall and this allows Bars E and F in *Fig.* 6.11 to be spaced out as shown.

The weight of the outlet trough outside the main wall causes a cantilever moment on Section X–X in Fig. 6.10. There is a load of 242 lb. at a distance of 3 ft. 8 in. from the centre-line of the main wall and three loads totalling 708 lb. at a distance of 1 ft. 10 in.

$$M = 242 \times 44 = 10{,}650 \text{ lb.-in.}$$
$$708 \times 22 = 15{,}600 \text{ lb.-in.}$$
$$26{,}250 \text{ lb.-in.}$$

There is also a cantilever moment of $224 \times 44 = 9{,}860$ lb.-in. caused by the rolling load but this does not occur all round the tank and cannot be dealt with by supplying a tension ring and a thrust ring. Assuming that the cantilever moment of 26,250 lb.-in. is taken on rings 36 in. apart,

$$\text{Tension} = \text{compression} = \frac{26{,}250}{36} = 729 \text{ lb. per ft.}$$

$$\text{Ring tension} = 729 \times 37{\cdot}5 \text{ ft.} = 27{,}400 \text{ lb.}$$

$$\text{Ring steel required} = \frac{27{,}400}{12{,}000} = 2{\cdot}28 \text{ sq. in.}$$

Provide six $\frac{3}{4}$-in. diam. bars = 2·65 sq. in.
These are marked G and H in *Fig.* 6.11.

The cantilever moment from the rolling load must run down to the bottom of the wall but will tend to spread along the circumference.

If it is assumed that the total cantilever moment spreads over 10 ft. at the base of the wall, the moment is $\frac{1{,}120 \text{ lb.} \times 44 \text{ in.}}{10 \text{ ft.}} = 4{,}930$ lb.-in. per foot.

This must be added to the restraint moment of 32,800 lb.-in., giving a total of 37,730 lb.-in.

On an 8-in. slab, $\frac{M}{bd^2} = \frac{37{,}730}{12 \times 8^2} = 49$. The total cantilever moment on Section X–X is 26,250 + 9,860 = 36,110 lb.-in. Using $\frac{1}{2}$-in diam. bars with $1\frac{1}{2}$-in. concrete cover,

$$A_{st} \text{ at base of wall} = \frac{37{,}730}{12{,}000 \times 0{\cdot}865 \times 6{\cdot}25} = 0{\cdot}58 \text{ sq. in.}$$

Provide $\frac{1}{2}$-in. diam. bars at 4-in. centres. These are bars A and B in *Fig.* 6.11, and these are continued by bars C and D to cover the moment on Section X–X in *Fig.* 6.10.

The load on the main wall is carried on a width of 3 ft. To bring the load central on this width a tailing-down moment is required. At the base of the vertical wall there is a total vertical load of 2,124 lb. which occurs 4 in. outside the centre-line of the wall, a small load from the toe of 270 lb. acting at a point 9 in. away, and a moment of 37,730 lb.-in. or 3,140 lb.-ft.

$$\text{Total} = 2{,}124 \times 0{\cdot}33 = 710 \text{ lb.-ft.}$$
$$270 \times 0{\cdot}75 = 200 \text{ ,, ,,}$$
$$3{,}140 \text{ ,, ,,}$$
$$4{,}050 \text{ lb.-ft.}$$

The total load on the floor, including self-weight is about 740 lb. per sq. ft. If the tailing-down length is x ft.,

$\frac{1}{2} \times 740x^2 = 4{,}050$ or $x = 3{\cdot}31$ ft., or 2,450 lb. at a distance of 1·65 ft.

The total load on the base 3 ft. wide is

$$2{,}124 + 270 + 2{,}450 = 4{,}844 \text{ lb. or } 0{\cdot}72 \text{ tons per sq. ft.}$$

This completes the preliminary design for the main reinforcement. The restraint steel required to resist external earth pressure when the tank is empty will be less

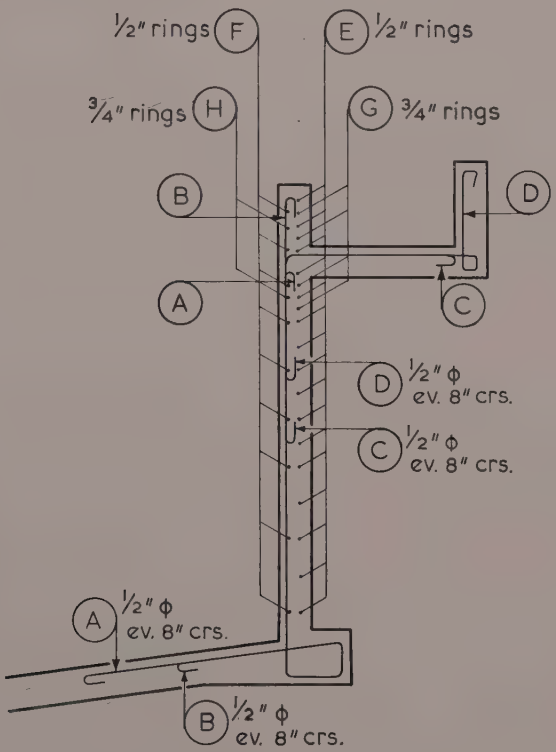


Fig. 6.11.

than one-quarter of the steel near the inside face of the wall and ½-in. diam. bars at 12-in. centres will be ample.

In preparing a final design it must be remembered that the radius of the main wall is 37 ft. 10 in. while the radius of the outside wall of the overflow trough is 41 ft. 6 in. For every foot length of main wall there is 1·095 ft. of outer wall and the load of 242 lb. in *Fig.* 6.10 should really be 265 lb., etc. There is a close spacing of vertical bars where bars A, B, C and D overlap. If bars G and H are welded into continuous rings there is plenty of room for them, but if they are hooked at the ends and lapped there is not quite enough room for the hooks, which would be normally six diameters or 4½ in. wide overall. Increasing the thickness of the main wall and floor to 9 in. and using ⅝-in. diam. vertical bars instead of ½-in. diam. would be an improvement. However, the tank would need to be completely re-designed, since the value of $H/\sqrt{TR}$ would be altered.

Walls with Sliding Joints

The introduction of a sliding joint at the base of a circular wall (where such a joint is permissible) reduces the design to simple arithmetic and this is left to the reader.

Interrupted Circular Walls

It has so far been assumed that the tank wall forms a complete circle. Where the wall is broken by the introduction of a large penstock or tank-to-tank flume it may not be possible to utilize hoop tension and the wall must cantilever.

CHAPTER SEVEN

RESTRAINT IN LARGE SHALLOW RECTANGULAR TANKS

THE walls of large shallow rectangular tanks will normally consist of uniform or tapered slabs cantilevering from the floor. Midway between the corners the walls are free to take up their primary deflection, but at the corners there is theoretically a restraint effect. In *Fig.* 7.1, a vertical section and a plan section at the corner of a cantilevered slab wall are shown. At any depth h the cantilever deflects outward by an amount Δ, but this outward deflection cannot take place at the corner, and at these points horizontal bending moments and shearing forces will occur, reaching maximum values at each corner. The calculated values of the bending moments and shearing forces, assuming that the corner remains exactly in its unloaded position,

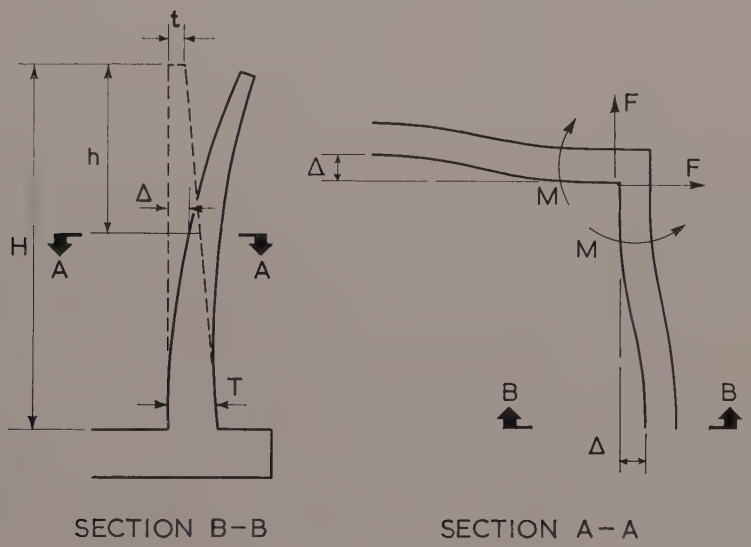


Fig. 7.1.

are shown in *Figs.* 7.2 and 7.3. The fact that each wall exerts a tension F on the other tends to pull the corner outwards and relieves the restraint moment and shear to some extent. The amount of relief is difficult to calculate as it depends on the shear modulus of the concrete. Judging by the problem of restraint in circular tanks it may be 10 to 20 per cent.

The restraint moment dies away as the distance from the corner increases and finally develops into a small reversed moment, equal to about 0·21 times the restraint moment. The moment changes sign at a distance of $\frac{0{\cdot}8F}{wh}$ from the corner.

If a cantilever wall 15 ft. high is required to retain water and the base section is to be designed for a resistance moment of $60bd^2$, then

$$60 \times 12 \times d^2 = \tfrac{1}{6} \times 62{\cdot}5 \times 15^3 \times 12.$$

Thus $d = 24$ in., say.
Reducing this to 8 in. at the top, a taper of T to $0{\cdot}333T$ is obtained.
The maximum restraint moment (*Fig.* 7.2) occurs at about $0{\cdot}4H$ (that is, about

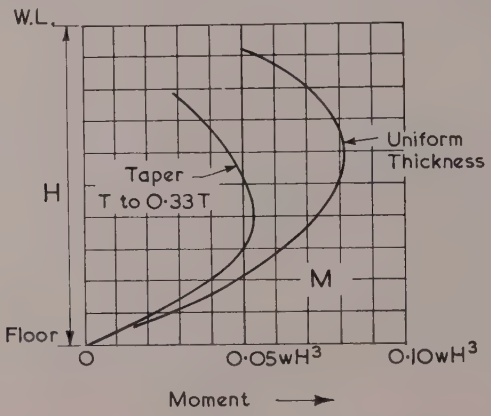


Fig. 7.2.

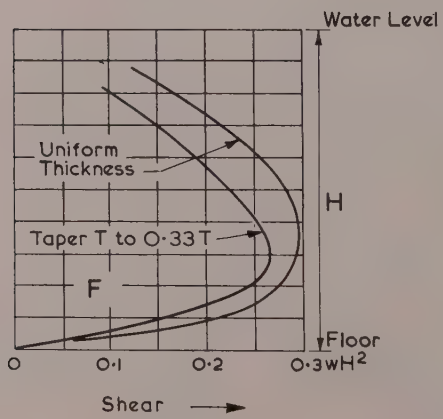


Fig. 7.3.

6 ft.) above the floor, where the wall is about 17·5 in. in thickness and is $0{\cdot}052wH^3$

$$= 0{\cdot}052 \times 62{\cdot}5 \times 15^3 = 11{,}000 \text{ lb.-ft.}$$
$$= 132{,}000 \text{ lb.-in. per foot height of wall}$$

$$\frac{M}{bd^2} = \frac{132{,}000}{12 \times 17{\cdot}5^2} = 36.$$

Shear at $0{\cdot}4H$ above floor (*Fig.* 7.3) $= 0{\cdot}25wH^2$

$$= 0{\cdot}25 \times 62{\cdot}5 \times 15^2 = 3{,}520 \text{ lb.}$$

Theoretically then, the horizontal steel required at this depth is

$$A_{st} = \frac{132{,}000}{12{,}000 \times 0{\cdot}9 \times 14{\cdot}5} = 0{\cdot}843 \text{ sq. in.}$$

$$+ \frac{3{,}520}{2 \times 12{,}000} = 0{\cdot}147 \text{ sq. in.}$$

$$0{\cdot}990 \text{ sq. in.}$$

Say $\frac{3}{4}$-in. diam. bars at $5\frac{1}{2}$-in. centres.

The moment dies away to zero at $\frac{0{\cdot}8F}{wh} = \frac{0{\cdot}8 \times 0{\cdot}25wH^2}{w \times 0{\cdot}6H} = 0{\cdot}333H$ or 5 ft.

A detail of the corner as calculated in the foregoing, is shown in *Fig.* 7.4, but a

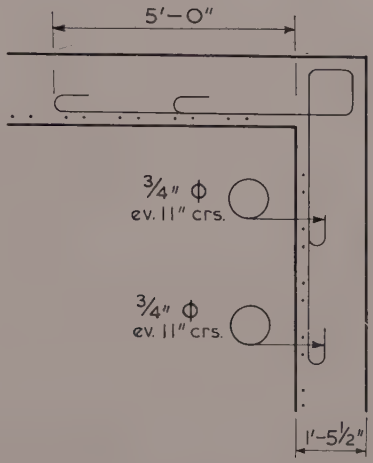


Fig. 7.4.

difficulty now arises. It is known that close attention must be paid to the problem of restraint at the base of a circular wall reinforced in hoop tension, because this occurs at a major horizontal construction joint and where the water pressure is a maximum. If leaks occur in the wall of a tank they are most likely to occur at horizontal construction joints, particularly at the bottom one. Leakage through vertical construction joints is rare, and at the junction of two straight walls there should be no construction joint. (The specification should stipulate this.) Despite the foregoing theory the writer knows of no case where trouble has arisen due to this corner restraint. All practical designers put steel round the inside of the corner, but few of them calculate this and the amount is usually comparable with the horizontal distribution steel in the wall. In this case if 0·15 per cent. of 12 in. by 17·5 in. is provided, 0·314 sq. in. is used, against a calculated amount of 0·99 sq. in. This latter figure is based on the maximum moment and with a slab of this thickness it may be averaged out to some extent. An area of one-half of the calculated maximum provided all the way up the corner would be a reasonable solution; that is, $\frac{5}{8}$-in. diam. bars at 7-in. centres over the full height of 15 ft.

CHAPTER EIGHT

MASS CONCRETE WALLS

Mass concrete gravity walls are commonly used for very small or very large heads of water. Those for very large heads of water include dock walls and gravity dams which are not usually classed as reservoir walls and are not discussed here.

Small walls are often found on small contracts where only the minimum plant and supervision are available and their design should include a factor of safety sufficiently large to cover these conditions.

Site Conditions—Penetration of Water

All foundation design must include a study of ground water conditions, as they exist before construction begins and as they may become after completion of the project. Loss of stability in gravity dams due to infiltration of water under pressure below the foundations or into horizontal construction joints has always been a grave cause for anxiety and has given rise to much controversy.

Some existing walls have a layer of asphalt on the water face protected by a $4\frac{1}{2}$-in. or 9-in. brick facing. The asphalt could be replaced by a layer of synthetic fabric glued to the concrete with synthetic adhesive. Some of these fabrics wrinkle badly, which is a distinct disadvantage in such circumstances. This waterproof layer should prevent water from penetrating the construction joints in the wall. On wet sites there is danger of outside water finding its way through the construction joints (or through patches of porous concrete) and blowing the waterproof layer and brick facing into the tank when it is empty.

Unbanked Gravity Walls on Dry Sites

On high ground (such as at the top of a chalk hill) upward water pressure under the base may usually be ignored. Assuming that the concrete weighs 140 lb. per cu. ft., the following two conditions must both be satisfied:

1. That the wall (when unsupported by external earth banking) must have a factor of safety of 2 against overturning about the outside toe.
2. That the line of resultant thrust must lie everywhere within the 'middle-third' of the horizontal section.

If these assumptions are made the three elementary wall sections shown in *Figs.* 8.1, 8.2 and 8.3 are obtained. All these sections have a factor of safety against overturning of at least 2 and in all cases the resultant falls on the middle-third point. The section shown in *Fig.* 8.2 requires only half the concrete needed for the section in *Fig.* 8.1. The section in *Fig.* 8.3 is for a division wall to take unbalanced pressure from either side. In all cases w has been taken as 62·5 lb. per cu. ft.

In *Fig.* 8.4 a preliminary design for a wall to retain 6 ft. of water is shown. Instead of being sloped, the back of the section is brought up in three lifts of 2 ft. 3 in.

Basing the design on *Fig.* 8.2 the design is commenced by drawing the line *ab*

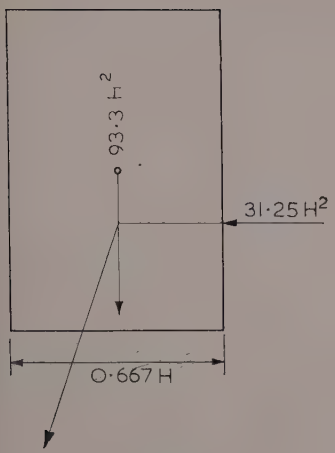


Fig. 8.1.

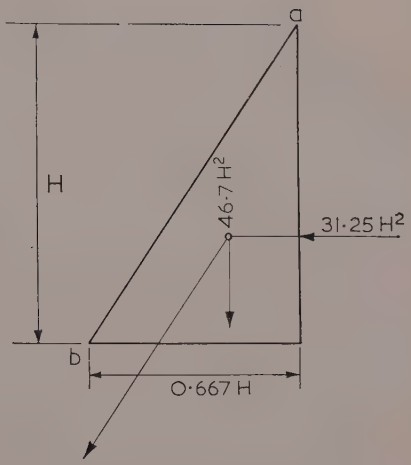


Fig. 8.2.

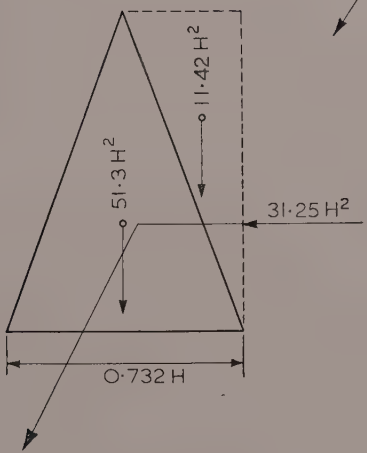


Fig. 8.3.

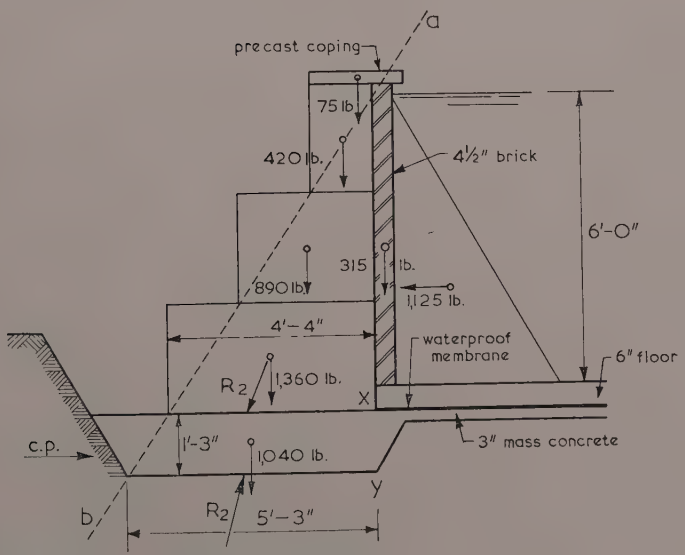


Fig. 8.4.

at an inclination of 1 vertical to 0·667 horizontal. The weights of the various lifts of the wall are shown in the figure in pounds per foot run of wall. Taking moments about point x it is found that R cuts the section 2·67 ft. from x, which is about 2·7 in. inside the middle-third point. Taking moments about y it is found that R_2 cuts the base 2·78 ft. from y or only about 2 in. from the centre point of the base, the pressure on the ground varying from 930 to 630 lb. per sq. ft. If all the horizontal force of 1,125 lb. were resisted by shear-friction under the base, this would amount to only 214 lb. per sq. ft. Before the wall could move, counter-pressure would develop between the base slab and the excavation where shown in *Fig.* 8.4 (assuming the base slab is cast against the face of the excavation). Since the waterproof membrane is below the 6-in. floor a head of water of 6 ft. 6 in. instead of 6 ft. should be considered and if there is a real danger of the water level rising accidentally to the level of the coping the stability should be checked under a head of 7 ft. Since the water will penetrate the $4\frac{1}{2}$-in. brick facing it might be thought that only the submerged weight of the brick would be effective. There will, however, be the pressure of a 6-ft. head of water on a strip $4\frac{1}{2}$ in. wide under the wall which, added to the submerged weight, brings it back to the full weight. The 6-in. floor will bring a certain amount of weight on to the inside edge of the footing slab thus increasing its stability.

Unbanked Gravity Walls on Drained Sites

If there is a danger of water finding its way under the wall from leaks in the floor and building up a pressure head, but the area round the tank is well drained, it may be assumed that the upward water pressure under the base varies from the full head of water in the tank on the line of the water face to zero on the outside

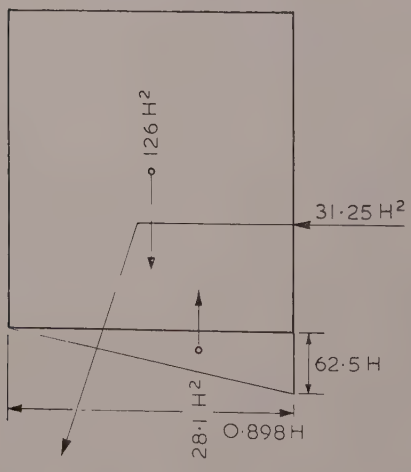


Fig. 8.5.

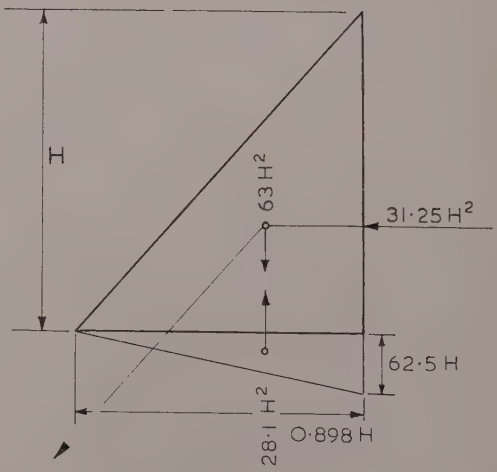


Fig. 8.6.

face. Making this assumption and keeping the line of thrust within the middle-third the three elementary wall sections shown in *Figs.* 8.5, 8.6 and 8.7 are obtained. These sections all have a factor of safety of a least 2 against bodily overturning, in the sense that the horizontal pressure can be increased to $62{\cdot}5H^2$ providing the upward pressure under the base remains at the values shown in the figures. In

Fig. 8.8 a preliminary design is shown for a division wall to retain 10 ft. of water, assuming upward water pressure under the base and at the construction joints. The design is commenced by taking *Fig.* 8.7 and drawing two lines *cd* and *cd'*

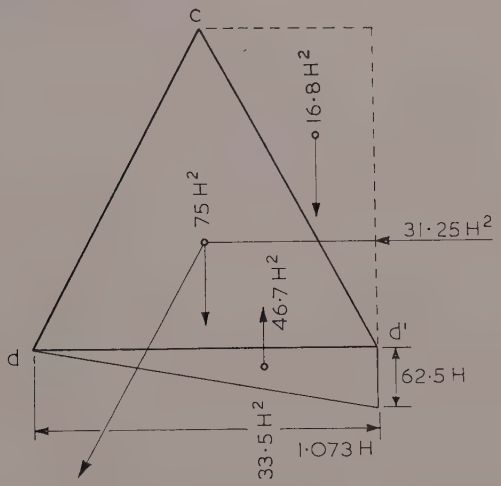


Fig. 8.7.

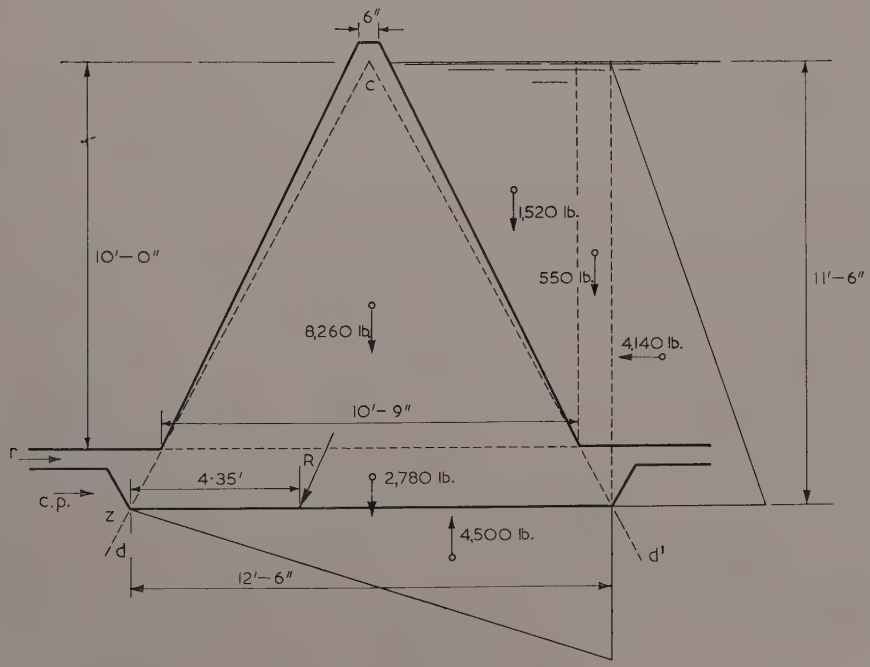


Fig. 8.8.

at such a slope that the base is 1·073 times the height. The head of pressure under the base could amount to 11·5 × 62·5 or 718 lb. per sq. ft. On a base 12 ft. 6 in. in width this amounts to 4,500 lb. per foot run of wall. The weights of the wall,

the footing slab and the water standing over the wall are as shown in the figure. Taking moments about point z,

8,260 × 6·25	= 51,600 lb.-ft.	
1,520 × 10·0	= 15,200 ,, ,,	
550 × 12·06	= 6,630 ,, ,,	
2,780 × 6·25	= 17,350 ,, ,,	
13,110 lb.	90,780 lb.-ft.	
−4,140 × 3·83 = 15,850		
−4,500 × 8·33 = 37,500	53,350 lb.-ft.	
	37,430 lb.-ft.	

Net weight = 13,110 − 4,500 = 8,610 lb.

Distance of R from $z = \frac{37{,}430}{8{,}610} = 4{\cdot}35$ ft.

This is about 2 in. inside the middle-third of the base, which is 12 ft. 6 in. in width. The wall is prevented from sliding by a reaction r from the floor slab and by counter-pressure from the ground marked *c.p.* in *Fig.* 8.8.

Unbanked Gravity Walls on Wet Sites

On a clay site where drainage is difficult the pressure head of water under the foundation could build up to the full value of the static head across the whole width of the foundation. The section shown in *Fig.* 8.5 would still hold good but the width of the base in *Fig.* 8.6 would need to be increased to $1{\cdot}16H$ and the base in *Fig.* 8.7 to $1{\cdot}115H$ in width to keep a factor of safety of 2 against bodily overturning. On such a bad site a tank with a thin floor would scarcely be practicable.

Banked Walls

Many existing service reservoirs really consist of a hole or embankment (or a combination of both) lined with puddle clay. The reservoir has a floor of concrete or brick paving and there is a wall, often circular in plan, of concrete or brick or concrete faced with brick. The floor and walls protect the soft clay and keep the inside of the reservoir clean. The walls should be strong enough to restrain the wet clay as it is puddled behind them but need not be watertight or strong enough to resist water pressure. Old reservoirs on puddle-clay foundations present a problem when they have to be roofed over. Where they are circular on plan the problem can be solved by adding a domed roof of aluminium alloy (see *Fig.* 27.34). Puddle clay is no longer in general use in this country.

When a bank of sand or ballast is used to support a mass concrete wall the wall must be watertight. However, when a bank of clay is used a watertight wall may permit the bank to dry and shrink in the summer leaving the top 6 ft. of the wall unsupported, and it is better either to supply a self-supporting watertight wall or revert to the old idea of well-packed clay and provide a wall sufficiently porous to keep the clay moist. In all cases the wall must be strong enough to resist the external earth pressure when the tank is empty. Assuming the earth pressure is 30 lb. per sq. ft. per foot of depth, the minimum width of base to give a factor of safety of 2 against overturning (assuming the soil weighs 100 lb. per cu. ft.) is as

shown in *Fig.* 8.9 for the three elementary shapes of wall section; these assume that the wall is straight. Many old reservoirs are circular in plan, which means that the brick or concrete lining is in a state of simple ring compression under pressure from the puddle clay when the reservoir is empty.

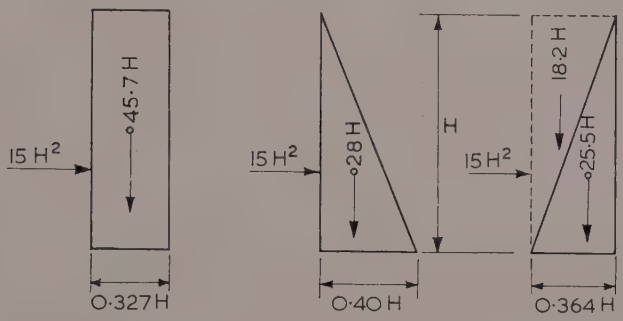


Fig. 8.9.

Mass Concrete Footing Slabs

On soft ground the design of the wall may require an extended toe to spread the load over a larger area and reduce the intensity of loading. That part of the toe which projects beyond the face of the main wall must act as a cantilever (see *Fig.* 8.10). If the amount of projection is L and the intensity of pressure is p, the

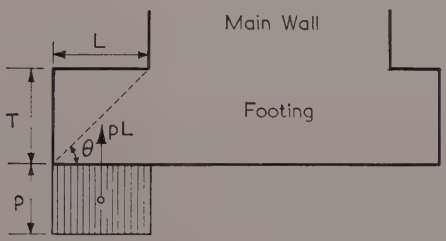


Fig. 8.10.

cantilever moment is $0{\cdot}5pL^2$. If the thickness of the toe is T then the tensile stress in bending is $0{\cdot}5pL^2 \times \frac{6}{T^2}$ assuming that all the terms are expressed in the appropriate units. If $p = 3$ tons per sq. ft., $L = 1$ ft. and $T = 1{\cdot}5$ ft. then the tensile stress is $0{\cdot}5 \times 3 \times 1^2 \times \frac{6}{1{\cdot}5^2} = 4$ tons per sq. ft. or 62·2 lb. per sq. in. If the tensile strength of the concrete is one-tenth of its compressive strength and a factor of safety of 2 is required the concrete must have a minimum test cube strength of $62{\cdot}2 \times 20 = 1{,}250$ lb. per sq. in. This and other values are shown in *Fig.* 8.11. When the intensity of loading exceeds 4 tons per sq. ft. the shear stress in the concrete should also be checked. This is not necessary for the values included in *Fig.* 8.11. In practice the value of p is usually a maximum at the edge of the toe while the values in *Fig.* 8.11 have been calculated for a uniform value of p. For the small walls being discussed here the difference is negligible. The full depth

of the footing slab should be cast in one continuous operation. The old rule-of-thumb said that the angle θ in *Fig.* 8.10 should not be less than 60 deg. (some designers said 45 deg.). These lines are shown for comparison in *Fig.* 8.11.

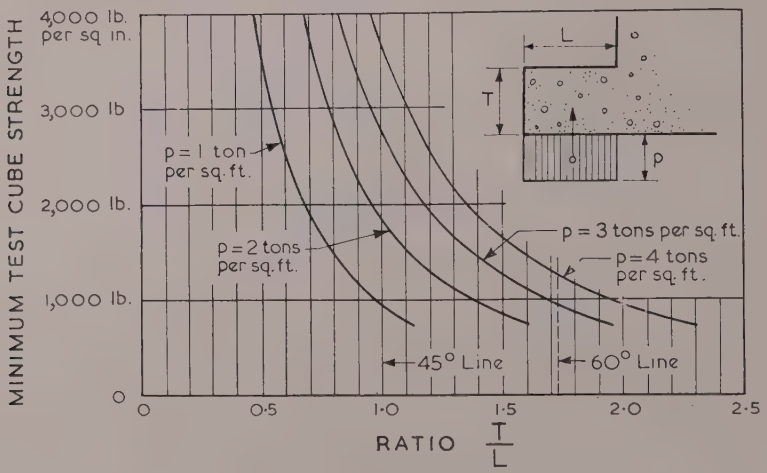


Fig. 8.11.

Mass Concrete Counterfort Walls

In a gravity wall the weight is most effective if applied at or near the water face. Any weight applied behind the middle-third, in theory at any rate, causes tension on the water face.

A preliminary design for a mass concrete counterfort wall 20 ft. high is shown

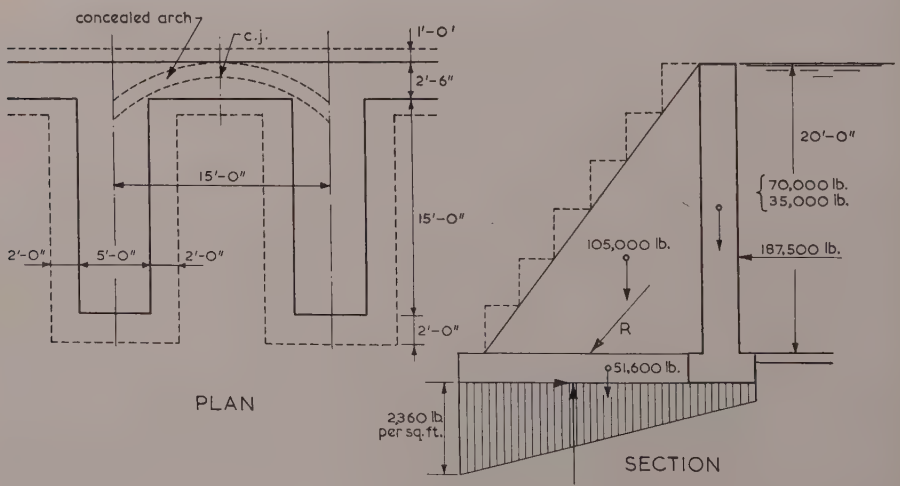


Fig. 8.12.

in *Fig.* 8.12. The counterforts are 5 ft. in thickness, are spaced at 15-ft. centres and are triangular in elevation. The 'slab' is 2 ft. 6 in. in thickness and is sufficiently thick to contain a concealed arch, to exploit which the corners of the reservoir

must be held securely (see *Fig.* 14.2). The maximum pressure on the ground, assuming that the total load is carried on an area of 9 ft. by 20 ft. 6 in. under the counterfort, is little more than 1 ton per sq. ft. and the shear-friction stress on the ground immediately below the footing slab is less than 0·5 ton per sq. ft.

A solid concrete wall would need to be at least 13 ft. 4 in. wide at the base (see *Fig.* 8.2). Assuming it were 1 ft. 6 in. wide at the top it would require (including the footing slab) 100 cu. yd. of concrete per 15-ft. length against 71 cu. yd. required from the wall shown in *Fig.* 8.12, but the solid wall would need less shuttering. The back of the counterfort in *Fig.* 8.12 would generally be brought up in vertical lifts, as shown by the broken lines. The shear strength at the horizontal construction joints in mass concrete work may be largely a matter of friction. Assuming that all the horizontal pressure of 187,500 lb. in *Fig.* 8.12 is taken on the area 5 ft. by 17 ft. 6 in. at the base of the wall, the ratio of horizontal to vertical load on the bottom construction joint is $\frac{187{,}500}{(105{,}000 + 35{,}000)} = 1{\cdot}34$. The coefficient of friction, concrete-on-concrete, is 2·0 to 3·0. To ensure that the slab and counterfort work together, construction joints in each lift of concreting should be made only at points midway between counterforts where marked *c. j.* in *Fig.* 8.12.

With sections of this thickness a concrete mix of 1 : $2\frac{1}{2}$: 5 should make a watertight wall.

CHAPTER NINE

CANTILEVERED SLAB WALLS

The most common type of cantilevered slab wall is that shown in *Fig.* 9.1. It can only be used on a dry or well-drained site as it depends for its stability on the

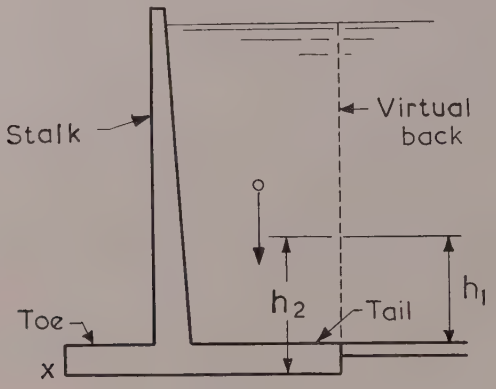


Fig. 9.1.

weight of water standing over its tail. Leakage of water sufficient to build up a pressure head under the base of the wall will cause it immediately to overturn (see *Fig.* 16.33).

Theoretical Proportions

In practice the arithmetic is complicated by the thickness of the members and the weight of the concrete. If the theoretical wall shown in *Fig.* 9.2 with thin,

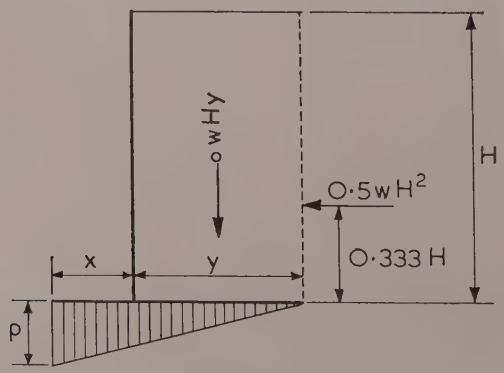


Fig. 9.2.

weightless members is considered, the width of base to satisfy any given conditions of stability can be calculated and the ratio between x and y to give the minimum width of base can be determined.

Assuming the wall must satisfy both of the following conditions:

1. That the factor of safety against bodily overturning about the outside end of the toe shall be not less than 2;
2. That the line of the resultant pressure on the foundation shall fall within the middle-third of the base;

the values given in *Fig.* 9.3 are obtained.

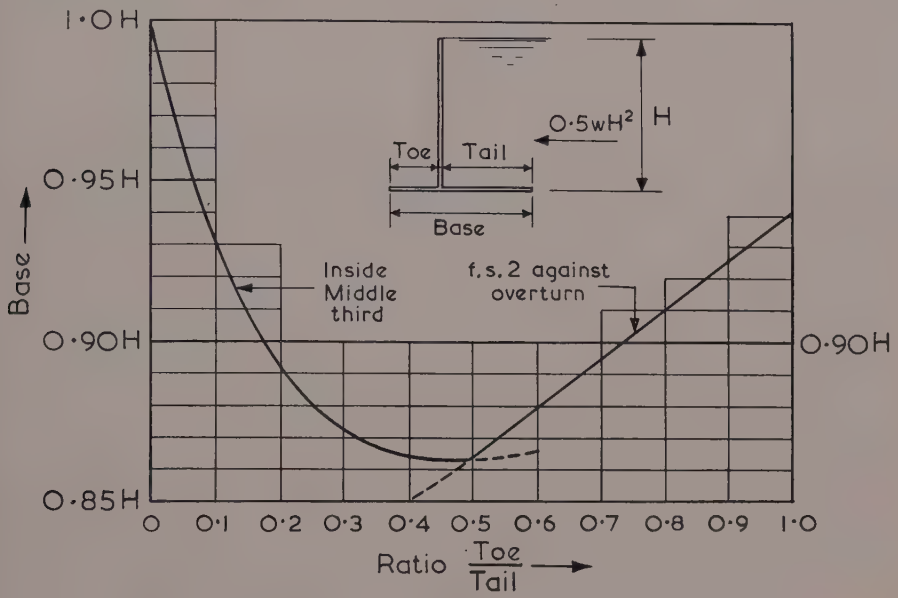


Fig. 9.3.

The minimum width of base occurs when the width of the toe is one-half of that of the tail and is 0·863 times the height *H*. With these proportions the line of the resultant falls exactly on the middle-third point, and the maximum pressure p on the foundation in *Fig.* 9.2 is 1·33*wH*. At the point where the stalk meets the base the three cantilever moments in the stalk, toe and tail must balance. With the width of base equal to 0·863*H* and the width of toe equal to one-half of the tail, these moments are as shown in *Fig.* 9.4.

Fig. 9.4.

Although the values in *Fig.* 9.3 may not give the best practical solution at the first attempt, they are always a useful guide for a preliminary design. Note that the proportions hold good for all values of w. Since overturning moments are taken about point X in *Fig.* 9.1 the water pressure acts at a height of h_2 above the

underside of the base where $h_2 = (0{\cdot}33 \times \text{depth}) + \text{thickness of base}$. The effective value of H in practice is therefore very nearly (depth of water plus thickness of base).

Thickness of Stalk

The moment per foot run of wall at any depth h below the surface is $0{\cdot}167wh^3$ and, working to a resistance moment of $60bd^2$, then with $b = 1$, $60d^2 = 0{\cdot}167wh^3$. Taking w as 62·5 lb. per cu. ft.,

$$60d^2 = 0{\cdot}167 \times 62{\cdot}5h^3 \quad \text{or} \quad d = 0{\cdot}416\sqrt{h^3},$$

where d is in inches and h is in feet.

If $h = 10$ ft., then $d = 0{\cdot}416\sqrt{1{,}000} = 13{\cdot}2$ in. The minimum thickness of wall required for a resistance-moment value of $60bd^2$ for a wall 20 ft. high is shown in *Fig.* 9.5. Few designers would make a watertight wall less than 5 in. in thick-

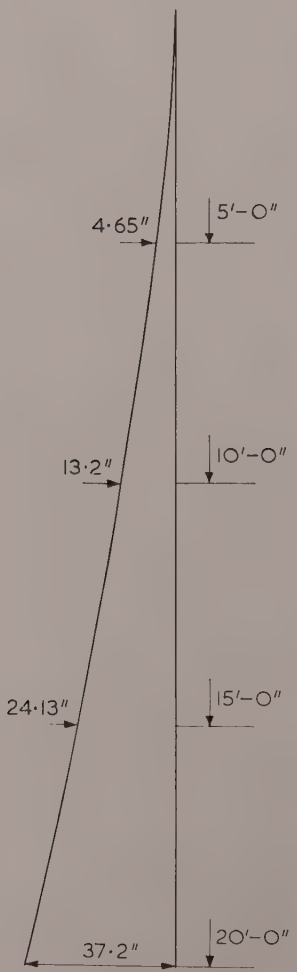


Fig. 9.5.

ness and most would not go below 6 in. At $60bd^2$, a 6-in. slab will carry a head of water of 5·92 ft.

For a wall 20 ft. high the wall section could therefore be any of the Shapes I, II, III, IV, V or VI shown in *Fig.* 9.6, and the shape selected should be most economical in terms of the combined cost of concrete, steel and centering. The area of section of each shape is given in the figure. The amount of concrete is obviously least for Shape VI and most for Shape I. Most modern designers would avoid Shape V. Up to a point the area of vertical steel will vary (very nearly) inversely as the thickness of wall, but the range over which this happens is very limited. Few designers would put less than 0·15 per cent. of vertical steel near each face of the wall and most would put not less than 0·3 per cent. near the water face. This proviso places a heavy handicap on Shape I where the wall is 37·2 in. in thickness throughout, for the vertical steel per foot run of wall must be not less than $0{\cdot}003 \times 12 \times 37{\cdot}2$ or 1·34 sq. in. near the water face and 0·67 sq. in. near the outside face all the way up. Since the earth pressure from a bank of dry earth can scarcely exceed 18 lb. per sq. ft. per foot of depth and a stress of 18,000 lb. per sq. in. may be employed to resist earth pressure (against 12,000 lb. per sq. in. to resist water pressure), the vertical steel near the outside face of the wall is dictated by the minimum of 0·15 per cent. and not by the moment due to earth pressure (see *Fig.* 9.10).

To resist a moment of $60bd^2$, about 0·7 per cent. of tension steel is required near the water face, provided the slab is made the minimum thickness. Assuming that 0·15 per cent. of vertical steel is provided near the outside face and 0·15 per cent. of horizontal steel near each face, the combined areas of steel are:

$$(0{\cdot}7 + 0{\cdot}15 + 0{\cdot}15 + 0{\cdot}15) = 1{\cdot}15 \text{ per cent. of } d.$$

If the thickness of the wall is doubled and a minimum of 0·3 per cent. of vertical steel is provided near the water face, the total reinforcement is now

$$(0{\cdot}3 + 0{\cdot}15 + 0{\cdot}15 + 0{\cdot}15) = 0{\cdot}75 \text{ per cent. of } 2d,$$
$$= 1{\cdot}5 \text{ per cent. of } d.$$

It therefore pays to keep as close as is reasonably possible to the minimum thickness given in *Fig.* 9.5.

The area of shuttering required is practically the same for all the shapes in *Fig.* 9.6. Most modern specifications rule out any shuttering system with a multiplicity of ties, in which case many standard wall forms may not be used. With well-designed timber shutters the difference in cost between the shuttering for the six shapes would be small. If the wall were sufficiently long to justify the provision of a travelling steel fit-up (see *Fig.* 26.8) the difference in cost between making the outer face curved or straight would be very small.

A curved or broken batter complicates the arrangement of steel, and the inner face, near which the heavy steel is provided, should normally be vertical or on a straight batter, the outer face being curved or bent.

Usually then, for a wall less than 6 ft. high a uniform thickness of 6 in. would be used; for a wall 10 ft. high a thickness of 6 in. at the top varying uniformly to 13·2 in. at the base (say 14 in. in practice). For a wall 15 ft. to 20 ft. high, use Shape III or Shape VI in *Fig.* 9.6.

The standard round end-hook on a bar is bent to a radius of two bar diameters and the hook on a $\frac{1}{2}$-in. diam. rod measures 3 in. overall; adding $1\frac{1}{2}$ in. on both sides gives a total of 6 in. This is the reason for showing Shape VI in *Fig.* 9.6. Reducing the top section to 6 in. (as in Shape IV) may introduce trouble with

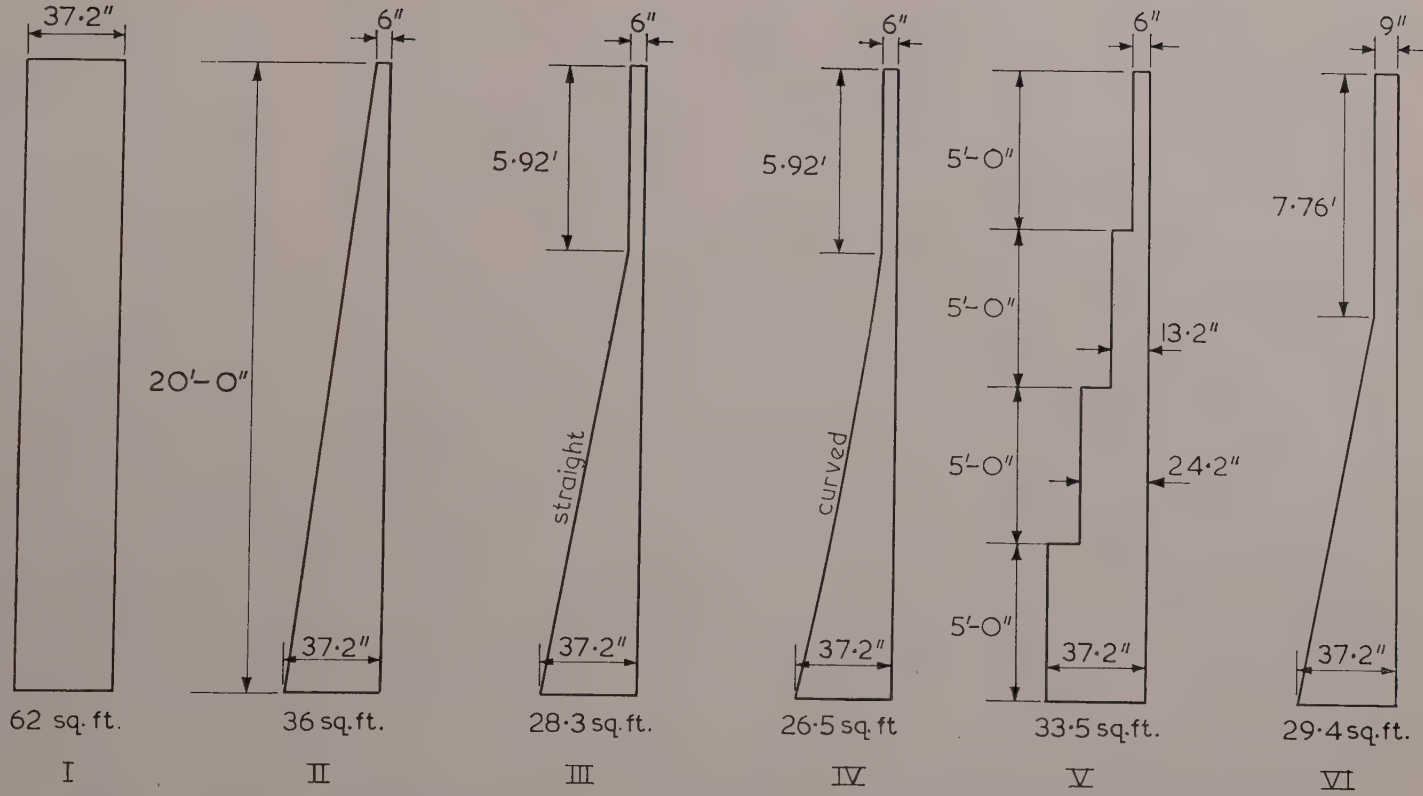

37·2″
62 sq. ft.
I
6″
20′–0″
37·2″
36 sq.ft.
II
6″
5·92′
straight
37·2″
28·3 sq.ft.
III
6″
5·92′
curved
37·2″
26·5 sq.ft
IV
6″
5′–0″
5′–0″
5′–0″
5′–0″
13·2″
24·2″
37·2″
33·5 sq.ft.
V
9″
7·76′
37·2″
29·4 sq.ft.
VI

Fig. 9.6.

the hooks on the main bars, whereas keeping to a minimum of 9 in. will avoid this. This point is emphasized in the worked example that follows.

The overall thickness of the slab is calculated from some value of M such as $60bd^2$ while the area of tension reinforcement, taking the maximum allowable stress as 12,000 lb. per sq. in., is calculated as

$$A_{st} = \frac{M}{12{,}000 \times (\text{lever arm})}.$$

The lever arm will vary with the percentage of reinforcement calculated on the area bd_1, where d_1 is the effective depth. The ratio between d_1 and d depends on the thickness of the slab, the amount of cover and the diameter of the rods.

With a 6-in. wall having $1\frac{1}{2}$ in. of concrete cover and reinforced with $\frac{1}{2}$-in. diam. rods, $d_1 = 4{\cdot}25$ in. and if $M = 60bd^2 = 60 \times 12 \times 6^2$, this gives

$$\frac{M}{bd_1^2} = \frac{60 \times 12 \times 6^2}{12 \times 4{\cdot}25^2} = 119{\cdot}5.$$

From *Fig.* 9.7, the lever arm is $0{\cdot}852d_1$.

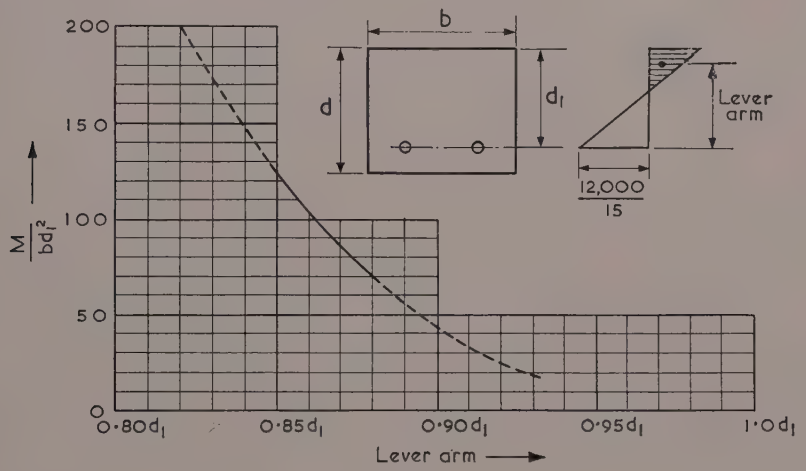


Fig. 9.7.

With a 24-in. wall having $1\frac{1}{2}$-in. concrete cover and reinforced with $\frac{3}{4}$-in. diam. bars, $d_1 = 22{\cdot}125$ in., and

$$\frac{M}{bd_1^2} = \frac{60 \times 12 \times 24^2}{12 \times 22{\cdot}125^2} = 70{\cdot}5.$$

From *Fig.* 9.7 the lever arm is $0{\cdot}88d_1$ or only some 3 per cent. larger. The values of interest in practical design lie on that part of the curve in *Fig.* 9.7 which is shown by the full line, and if the lever arm is assumed to be $0{\cdot}865d_1$ for all sections of the wall, this is sufficiently accurate in practice.

If a tapering wall section is adopted then, theoretically, it should be designed as a tapering beam but the taper is too small to make any practical difference in this case.

The shearing stresses in the stalk of the wall are not likely to be high. Working to a resistance moment of $60bd^2$, the thickness must be not less than $0{\cdot}416\sqrt{h^3}$, if $w = 62{\cdot}5$ lb. per cu. ft. Working to a permissible shearing stress of 115 lb. per

sq. in. and taking the lever arm as $0{\cdot}67d$, then d must be not less than $0{\cdot}034h^2$. At the point where $0{\cdot}416\sqrt{h^3} = 0{\cdot}034h^2$, $h = 150$ ft. Consequently, the shear stress in the stalk of the wall need not be checked for walls less than 150 ft. high.

Preliminary Design for Cantilevered Slab Wall 15 ft. High

The wall is to retain water and to be designed for $M = 60bd^2$. Taking all values per foot run of wall, total water pressure $= 0{\cdot}5 \times 62{\cdot}5 \times 15^2 = 7{,}030$ lb. M at base of stalk $= 7{,}030 \times 60 = 422{,}000$ lb.-in.

$$\text{Minimum } d \text{ at } 60bd^2 = \sqrt{\frac{422{,}000}{60 \times 12}} = 24{\cdot}2 \text{ in., say } 25 \text{ in.}$$

Starting with a section similar to Shape III in *Fig.* 9.6, the section is as shown in

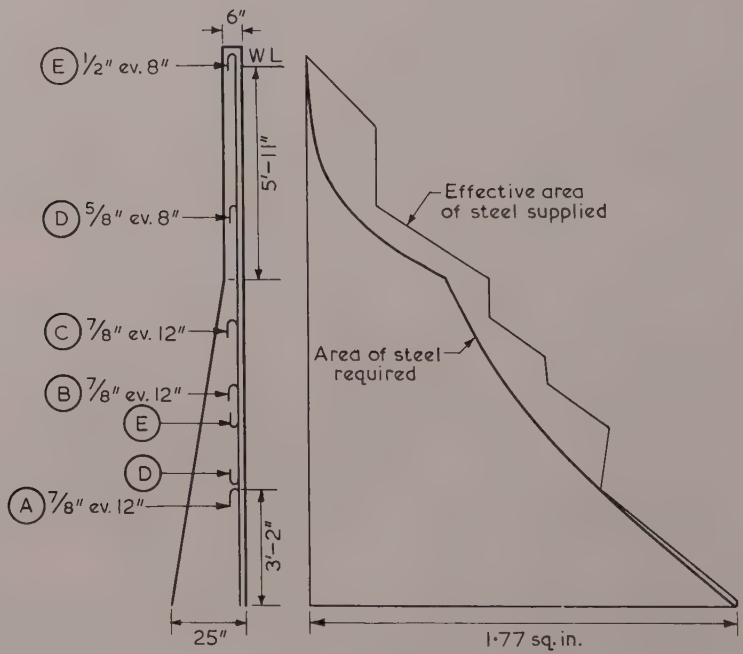


Fig. 9.8.

Fig. 9.8. The wall tapers 19 in. in a height of 9 ft. 1 in. At a depth of 10 ft. the wall is 14·55 in. in thickness and d_1 at this depth is, say, 12·55 in.

$$M \text{ at 10-ft. depth} = 0{\cdot}167 \times 62{\cdot}5 \times 10^3 \times 12$$
$$= 125{,}000 \text{ lb.-in.}$$
$$A_{st} = \frac{125{,}000}{12{,}000 \times 0{\cdot}865 \times 12{\cdot}55} = 0{\cdot}96 \text{ sq. in.}$$

Repeating the process for depths of 6 ft., 8 ft., 12 ft., 14 ft. and 15 ft., the curve in *Fig* 9.8 is obtained.

$$\text{At base, } A_{st} = \frac{422{,}000}{12{,}000 \times 0{\cdot}865 \times 23} = 1{\cdot}77 \text{ sq. in.}$$

If $\frac{7}{8}$-in. diam. bars at 4-in. centres are provided in three lengths A, B and C, bar C cannot be run any higher than shown, for a $\frac{7}{8}$-in. diam. standard hook is 5·25 in. wide and this will not go in a 6-in. wall. The hook could be bent hot to 3 in. overall (which would make it much less efficient) or it could be turned sideways (which would be very inconvenient to bend). Apart from these two makeshift solutions bar C can be stopped off where shown in *Fig.* 9.8 and other bars D and E introduced to supply the necessary area of steel in the upper part of the wall. The overall width of a hook for a $\frac{5}{8}$-in. diam. rod is $3\frac{7}{8}$ in. against a net width of 3 in. available in a 6-in. wall with $1\frac{1}{2}$-in. concrete cover. With a straight detached bar this can be overcome by rotating the bar through about 45 deg.

If the top of the wall is thickened to 8 in. and $\frac{3}{4}$-in. bars at 3-in. centres are used, as in *Fig.* 9.9, one bar J can be run right up to the top and no extra loose bars,

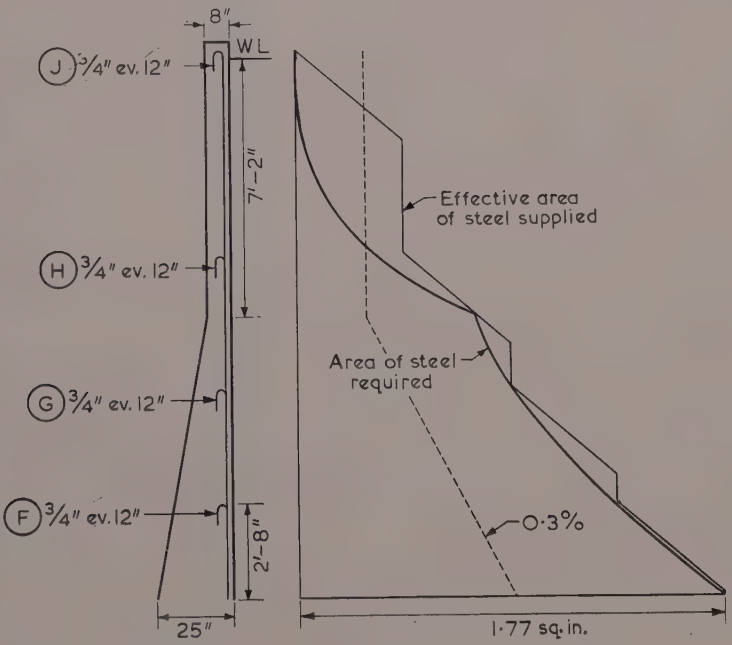


Fig. 9.9.

similar to bars D and E in *Fig.* 9.8, are needed. This not only makes the bar arrangement simpler and more foolproof but it reduces the weight of steel by eliminating the laps in the design in *Fig.* 9.8, saving about 0·09 cwt. per foot run of wall. Against this the wall in *Fig.* 9.9 requires 0·035 cu. yd. more concrete per foot run. If concrete costs 1·5 units per cu. yd. and steel costs 1·0 units per cwt., a small saving in cost and a sounder wall are obtained by adopting the design shown in *Fig.* 9.9.

If the spacing of the bars in *Fig.* 9.9 were increased from 3 in. to $3\frac{1}{2}$ in., the calculated stress would be increased from 12,000 to 14,000 lb. per sq. in. The author would accept this as sound design. Increasing the spacing to 4 in. would increase the calculated stress to 16,000 lb. per sq. in. Many existing walls have been designed to this figure.

If the wall in *Fig.* 9.9 is to be banked round (and this is usually done after the

reservoir is filled and tested), if the earth pressure is taken as 20 lb. per sq. ft. per foot of depth, and working to a steel stress of 18,000 lb. per sq. in., the area of steel required, taking the lever arm as $0{\cdot}92d_1$, will be about 0·2 times the steel area in *Fig.* 9.9; this is shown in *Fig.* 9.10. The area required to resist the moment due to earth pressure is less than the minimum permissible area of 0·15 per cent.

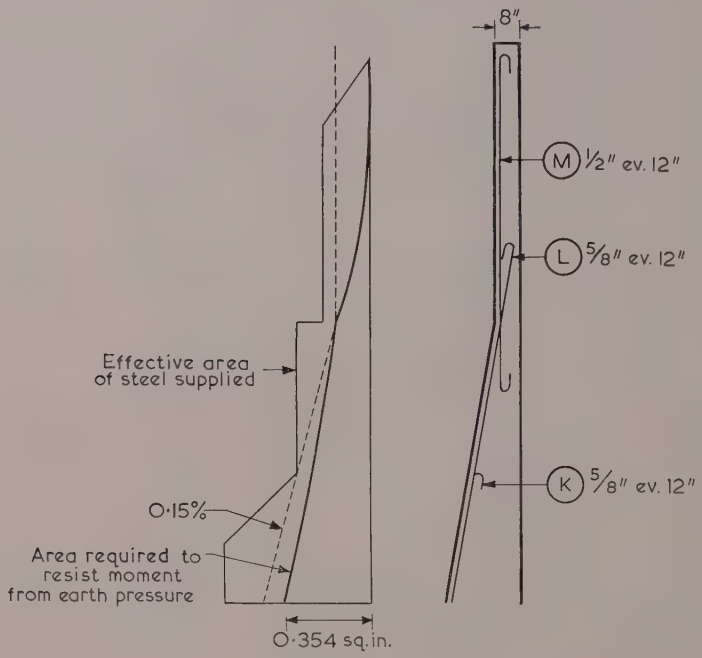


Fig. 9.10.

The total area of the section of the wall in *Fig.* 9.9 is 15·9 sq. ft. At 0·3 per cent. the total area of longitudinal distribution steel required is

$$\frac{0{\cdot}3}{100} \times 15{\cdot}9 \times 144 = 6{\cdot}85 \text{ sq. in.}$$

At the top of the wall $0{\cdot}003 \times 8 \times 12 = 0{\cdot}29$ sq. in. per foot height is required, say $\frac{1}{2}$-in. diam. rods at 16-in. centres near both faces. At the base of the wall $0{\cdot}003 \times 25 \times 12 = 0{\cdot}90$ sq in. is required, say $\frac{3}{4}$-in. diam. rods at 12-in. centres near both faces.

In practice the danger of vertical cracks developing increases from bottom to top of the wall, reaching a maximum at the exposed top edge, and more horizontal steel is required in the top of the wall than near the bottom. The author would prefer the arrangement in *Fig.* 9.11. This has a total sectional area of steel of 6·89 sq. in. (against the figure of 6·85 sq. in. calculated in the foregoing) but has a higher percentage of steel in the upper part of the wall than in the lower. This is another illustration of the fact that any Code of Practice that specifies fixed values for thicknesses and working stresses can never be more than a collection of inaccurate generalities. Even if the specified values are fair average figures they must be too high for certain site conditions and too low for others.

For a long wall on an exposed site the distribution bars in the upper part of the wall should be deformed bars or plain bars with long laps and end-hooks.

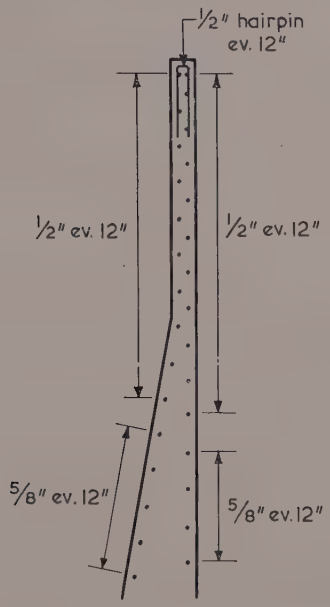


Fig. 9.11.

Design of Wall Base

Many early bases were designed as shown in *Fig.* 9.12. How many were actually built as designed and how many were modified on site to the form shown in *Fig.* 9.1 is not on record. To produce a strongly sloping top surface the contractor may

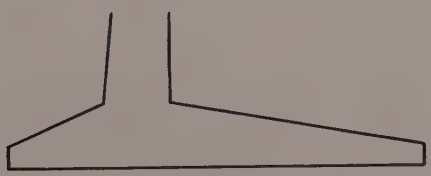


Fig. 9.12.

use over-dry lightly-tamped concrete without top shuttering or a proper mix properly consolidated and held down by top shutters. The first expedient gives a poor but cheap job and the second a good but expensive one, and modern bases are usually designed as shown in *Fig.* 9.1.

As the reservoir floor is usually laid to falls the top surface of the tail may slope about 1 in. in 10 ft.

Preliminary Design for Base for 15-ft. Wall

Taking the wall in *Figs.* 9.9, 9.10 and 9.11, and assuming that the width of the base is $0{\cdot}863H$ (see *Fig.* 9.3), first H must be determined. From *Fig.* 9.4 the theoretical moment in the tail is about 0·7 times the moment in the stalk, and

the tail at $60bd^2$ need only be about 0·84 times the thickness of the bottom of the stalk, or say 21 in. The effective value of H in *Fig.* 9.3 is about 15 ft. + 1 ft. 9 in., or 16·75 ft. (see also *Fig.* 9.1). The width of base, taking the minimum value of $0{\cdot}863H = 0{\cdot}863 \times 16{\cdot}75$ or say 14 ft. 6 in.

This corresponds to a toe-to-tail ratio of 1 : 2, so that the width of the toe is 4 ft. 10 in. and of the tail is 9 ft. 8 in. making up the total of 14 ft. 6 in.

The theoretical wall has no thickness but the actual wall varies from 8 in. to 25 in. and it must be decided from where the toe and tail are to be measured. Measuring from the outside face of the 8-in. upper portion, the dimensions in *Fig.* 9.13 are obtained. The loading per foot run of wall is shown in the figure.

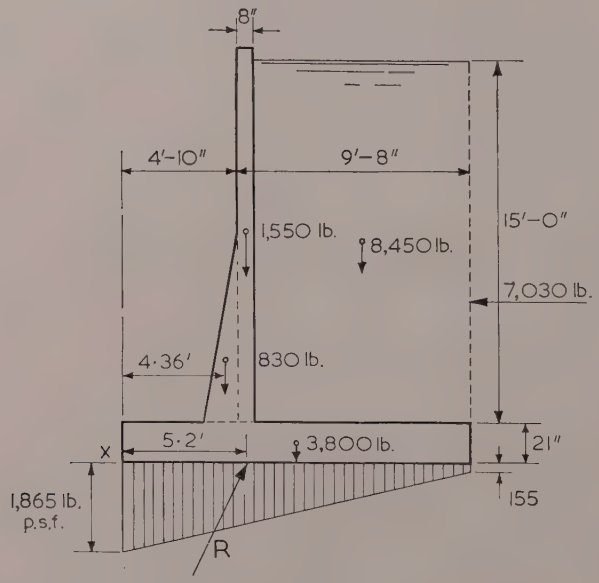


Fig. 9.13.

Taking moments about point X:

830 × 4·36	=	3,600	lb.-ft.
1,550 × 5·17	=	8,000	,, ,,
8,450 × 10·00	=	84,500	,, ,,
3,800 × 7·25	=	27,600	,, ,,
14,630		123,700	lb.-ft.
7,030 × 6·75	=	47,500	,, ,,
		76,200	lb.-ft.

Horizontal distance of R from point X $= \dfrac{76{,}200}{14{,}630} = 5{\cdot}2$ ft., or 2·05 ft. from the centre of the base.

$$p = \frac{14{,}630}{14{\cdot}5} \pm \frac{14{,}630 \times 2{\cdot}05 \times 6}{14{\cdot}5^2}$$
$$= +1{,}010 \pm 855$$
$$= +1{,}865 \text{ to } +155 \text{ lb. per sq. ft.}$$

Of the total upward pressure of 14,630 lb., 5,670 lb. acts under the toe and 6,170 lb. under the tail, as shown in *Fig.* 9.14.

Upward moment on toe = (5,670 × 1·78) − (895 × 1·71) = 8,570 lb.-ft.
Downward moment on tail = (8,450 × 4·5) + (2,360 × 4·5) − (6,170 × 3·34)
= 28,100 lb.-ft.

The cantilever moment at the bottom of the stalk is 7,030 × 5 = 35,150 lb.-ft. The sum of the moments in the toe and tail in the theoretical wall balances the moment in the stalk (see *Fig.* 9.4). In the present case, the corresponding moments are 8,570 + 28,100 (= 36,670 lb.-ft.) and 35,150 lb.-ft., which do not balance.

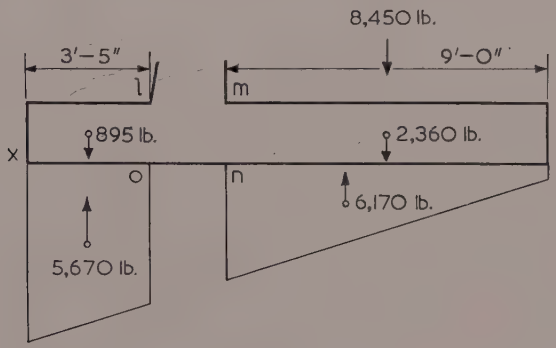


Fig. 9.14.

The discrepancy is due to the fact that the loading on the solid block *lmno* in *Fig.* 9.14 has been ignored.

The moment on the tail is 28,100 lb.-ft. = 337,000 lb.-in. and for a resistance moment of $60bd^2$ a thickness of 21·62 in. is needed, compared to the preliminary figure of 21 in.

The area of steel available consists of the bottom ends of bars F, G, H and J in *Fig.* 9.9, totalling 1·77 sq. in.
Making the base 22 in. in thickness,

$$A_{st} = \frac{337{,}000}{12{,}000 \times 0{\cdot}865 \times 20} = 1{\cdot}625 \text{ sq. in.}$$

The bar arrangement is as shown in *Fig.* 9.15.

If this wall is banked up to water level with earth weighing 100 lb. per cu. ft., which exerts a pressure of 20 lb. per sq. ft. per foot of depth, the loading per foot run of wall is as shown in *Fig.* 9.16.
Taking moments about point X, the horizontal distance of *R* from point X is 5·39 ft.
Of the total upward pressure of 12,867 lb., 5,670 lb. acts under the tail.

Upward *M* on tail = (5,670 × 3·49) − (2,360 × 4·50) = 9,180 lb.-ft.
= 110,000 lb.-in.

$$A_{st} = \frac{110{,}000}{18{,}000 \times 0{\cdot}92 \times 20} = 0{\cdot}332 \text{ sq. in.}$$

The lower ends of bars L and K in *Fig.* 9.10 are available, giving 0·61 sq. in. of steel.

The sectional area of the base is 14 ft. 6 in. by 1 ft. 10 in. or 26·6 sq. ft. If the area of distribution steel specified is 0·3 per cent., 11·45 sq. in. are required. Twenty-two bars are shown in *Fig.* 9.15. If these bars were $\frac{5}{8}$-in. in diameter the total

area would be 6·74 sq. in. or only 0·177 per cent. On some sites this would be enough. If $\frac{3}{4}$-in. diam. bars were used they would total 9·7 sq. in. or 0·254 per cent., which is sufficient for any slab of this thickness resting on the ground in a temperate climate. The shear on the ground under the base, ignoring counter-pressure against the front face of the toe, is 7,030 lb. on 14·5 sq. ft. or 485 lb. per

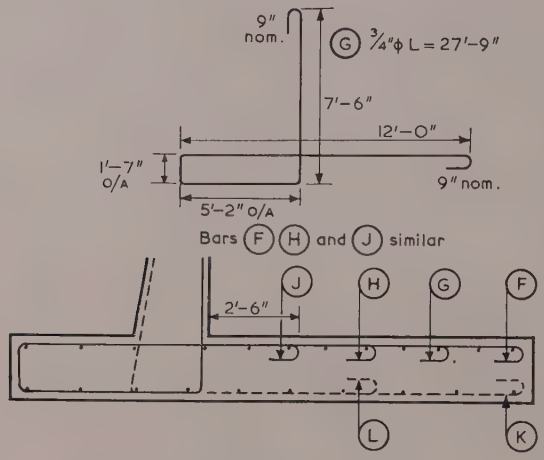


Fig. 9.15.

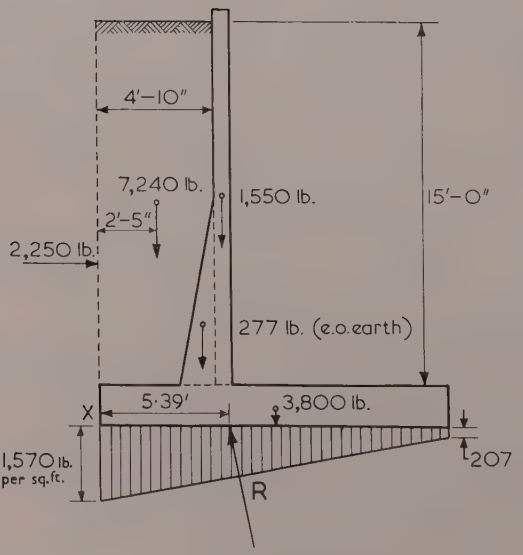


Fig. 9.16.

sq. ft. After the banking (if any) is placed, the maximum pressure on the ground is (1,865 + 1,570) lb. or 1·53 tons per sq. ft. These figures are safe on any site where this form of construction is possible.

This completes the preliminary design. The final design is arrived at by modifying this preliminary design to suit the site conditions.

Cantilevered Slab Division Wall

For a division wall formed as a cantilevered slab, since major vertical steel is required near both faces of the wall, the choice of section is practically restricted to Shapes VII, VIII and IX in *Fig.* 9.17. In *Fig.* 9.10 vertical steel is provided

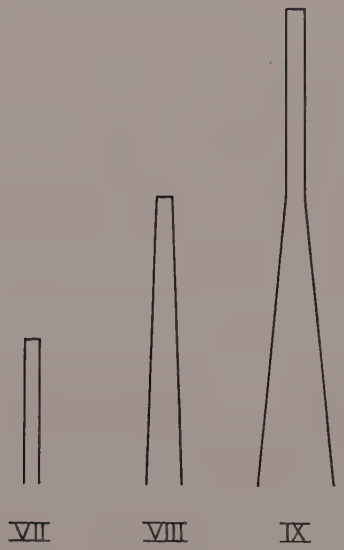


Fig. 9.17.

near the bent face of the wall but this only represents about one-fifth of the steel on the straight face and consists of relatively few small bars. To adopt a similar arrangement of numerous heavy bars near both faces of the wall leads to a jumble of crossing and overlapping bars at the point where the slope of the wall changes. For this reason Shape VIII would normally be used for a wall 15 ft. high and Shape IX for a wall having a height of 20 ft. or more, Shape VII being used for walls 6 ft. or 8 ft. in height.

Preliminary Design for Division Wall 15 ft. High

If Shape VIII is adopted and the wall is made 8 in. thick at the top, the section in *Fig.* 9.18 is obtained. The curve showing the area of steel required is calculated as before. At a depth of 10 ft. the wall is 19·51 in. in thickness or $d_1 = 17{\cdot}51$ in., say. The cantilever moment is 125,000 lb.-in.

$$A_{st} = \frac{125{,}000}{12{,}000 \times 0{\cdot}865 \times 17{\cdot}51} = 0{\cdot}688 \text{ sq. in.}$$

Keeping to a minimum of 0·3 per cent. of steel near the water face the bar arrangement shown in *Fig.* 9.18 is obtained. The arrangement would be improved if bar O were carried 2 feet further up the wall. If this were done, the author would be prepared to see the spacing of the bars at the base of the wall increased from 3 in. to $3\frac{1}{2}$ in., thus increasing the calculated stress in the steel from 12,000 to 14,000 lb. per sq. in. If the designer is prepared to accept a minimum of 0·15 per cent. of distribution steel near the water-retaining face, bar P could be shortened by about

5 ft. The area of section in *Fig.* 9.18 is 21·3 sq. ft. Using 0·3 per cent. of distribution steel, 0·003 × 21·3 × 144 = 9·2 sq. in. are required. If the horizontal bars are spaced at 12-in. centres, there are 32 bars. If the bottom 4 bars on each side are $\frac{3}{4}$-in. in diam., the next 6 are $\frac{5}{8}$-in. in diam. and the top 6 are $\frac{1}{2}$-in. diam., the

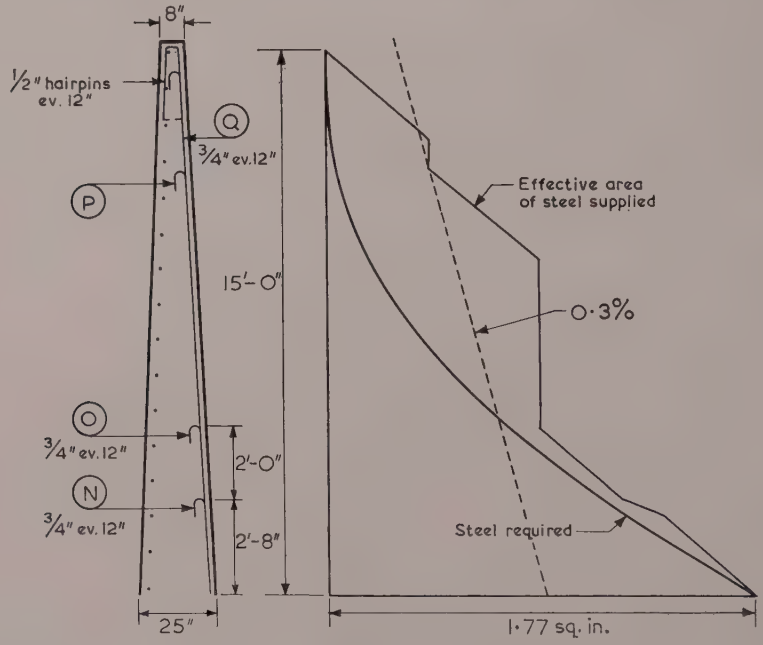


Fig. 9.18.

total steel area will be 9·4 sq. in. The laps should always be staggered, and not less than 40 diameters and never less than 2 ft. in length. If the reservoir has no roof and it may stand empty for some appreciable time the distributors in the upper part of the wall should have hooked ends or be deformed bars. Under these conditions which can cause heavy tension in the upper part of the wall the bars should

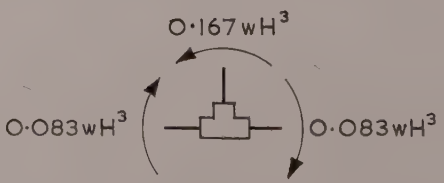


Fig. 9.19.

all be $\frac{5}{8}$-in. in diameter. If the tank is roofed over and is always in use, the spacing could be increased to 15 in. and the number of bars reduced to 26.

Since toe and tail are equal in width, a base of a width equal to 0·943 times the height is required for a thin weightless wall if a factor of safety of 2 against overturning is required (see *Fig.* 9.3).

In this theoretical wall the cantilever moment of $0{\cdot}167wH^3$ at the bottom of the stalk is resisted by two equal moments of $0{\cdot}083wH^3$ in the toe and tail, as shown in *Fig.* 9.19. For a resistance moment of $60bd^2$, the base slab therefore need only

be 0·707 times as thick as the bottom of the stalk. For the 15-ft. wall in *Fig.* 9.18 the base will be about 0·7 × 25 or, say, 18 in. in thickness and the effective height of the wall will be about 16 ft. 6 in. (see *Fig.* 9.1). The width of the base for the theoretical wall is 0·943*H* and a value of 0·943 × 16·5 ft. or 15 ft. 6 in. may be

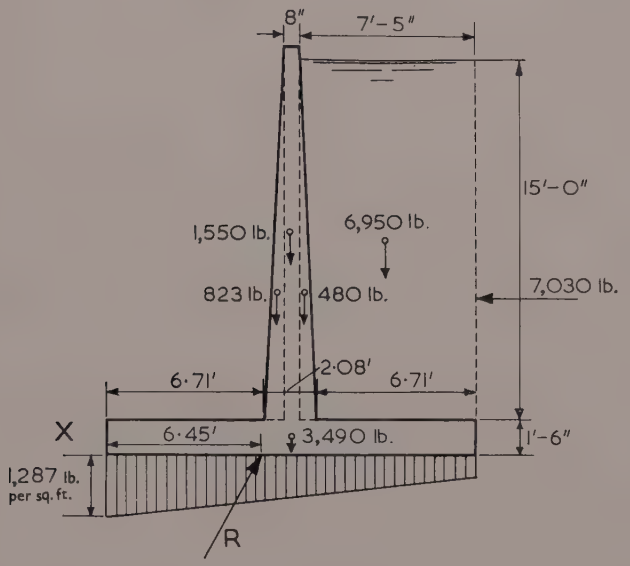


Fig. 9.20.

adopted for a preliminary design. This is shown in *Fig.* 9.20, which also shows the loading per foot run of wall. Taking moments about point X:

823 × 7·18		=	5,900 lb.-ft.
1,550 × 7·75		=	12,000 ,, ,,
480 × 8·32		=	4,000 ,, ,,
6,950 × 11·79		=	82,000 ,, ,,
3,490 × 7·75		=	27,100 ,, ,,
say 13,300			131,000 lb.-ft.
	7,030 × 6·5	=	45,600 ,, ,,
			85,400 lb.-ft.

Horizontal distance of *R* from X $= \frac{85,400}{13,300} = 6·45$ ft. This is 1·30 ft. from the centre-line.

$$p = \frac{13,300}{15·5} \pm \frac{13,300 \times 1·30 \times 6}{15·5^2} = +1,287 \text{ to } +425 \text{ lb. per sq. ft.}$$

Owing to the weight and thickness of the members the true factor of safety against overturning is $\frac{131,000}{45,000}$ or 2·91, against 2·0 in the thin weightless wall in *Fig.* 9.3. Of the total upward earth pressure of 13,300 lb., 7,360 lb. acts on the toe in front

of the stalk, 1,800 lb. is directly under the stalk and 4,100 lb. acts under the tail behind the stalk. The base projects 6·71 ft. in front of and behind the stalk and the weight of each projecting part is 6·71 × 225 or 1,510 lb.
The upward moment on the toe is

$$(7{,}360 \times 3{\cdot}55) - (1{,}510 \times 0{\cdot}5 \times 6{\cdot}71) = 21{,}080 \text{ lb.-ft.}$$

The downward moment on the tail is

$$(6{\cdot}71 \times 15 \times 62{\cdot}5 \times 0{\cdot}5 \times 6{\cdot}71) + (1{,}510 \times 0{\cdot}5 \times 6{\cdot}71) - (4{,}100 \times 3{\cdot}01) = 13{,}920 \text{ lb.-ft.}$$

The cantilever moment on the stalk is 7,030 × 5·0 = 35,150 lb.-ft.
The sum of the moments in the toe and tail is 21,080 + 13,920 = 35,000 lb.-ft. The small discrepancy is due to neglecting the loading on the solid block 25 in. wide and 18 in. deep at the intersection of the members.

The moment on the toe is 21,080 lb.-ft. = 252,200 lb.-in., with compression on the water face. At $250bd_1^2$, the minimum d_1 is only 9·17 in.

The moment on the tail is 13,920 lb.-ft. = 167,000 lb.-in. At $60bd^2$, the minimum d is 15·25 in. A preliminary design for the base is shown in *Fig.* 9.21.

$$A_{st} \text{ required near underside} = \frac{252{,}200}{18{,}000 \times 0{\cdot}865 \times 16} = 1{\cdot}01 \text{ sq. in.}$$

Bars N, O, P and Q from the stalk total 1·77 sq. in.

$$A_{st} \text{ required near top} = \frac{167{,}000}{12{,}000 \times 0{\cdot}865 \times 16} = 1{\cdot}01 \text{ sq. in.}$$

If bars S and T are $\frac{5}{8}$-in. diam. at 12-in. centres and bars R are $\frac{3}{4}$-in. diam. at 12-in. centres, the total area is 1·06 sq. in.

The area of section of the base is 18 in. by 15 ft. 6 in. or 23·2 sq. ft. For 0·3 per cent. the distribution steel required would be 10·0 sq. in. Twenty-eight bars are shown in *Fig.* 9.21. If these were $\frac{5}{8}$-in. in diameter they would total 8·6 sq. in. or 0·258 per cent. This is ample for such a thick slab resting on the ground.

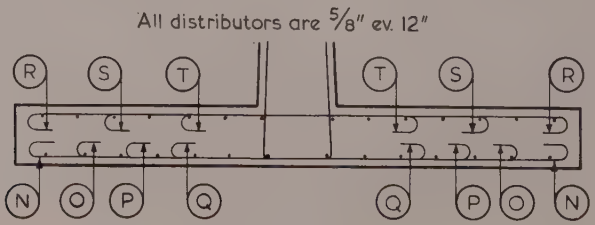


Fig. 9.21.

The bottom layer of bars in the centre of the base in *Fig.* 9.21 is rather crowded with pairs of bars at 3-in. centres. If $\frac{7}{8}$-in. diam. bars at 4-in. centres were substituted for $\frac{3}{4}$-in. diam. bars at 3-in. centres the base would be easier to concrete.

Joint with Floor

The preliminary designs for the walls have been prepared on the assumption that they were independent of the floor slab. The joint between walls and the floor may be made as shown in the upper detail in *Fig.* 9.22, or possibly as in the lower detail, where the piece of floor attached to the wall base acts as part of the

base. If the water is 15 ft. deep and the floor 6 in. in thickness it can cantilever 2 ft. as shown.

$$A_{st} = \frac{24{,}300}{12{,}000 \times 0{\cdot}865 \times 4{\cdot}25} = 0{\cdot}55 \text{ sq. in.},$$

say, $\frac{1}{2}$-in. diam. bars at 4-in. centres.

Shear stress $q = 46$ lb. per sq. in.

In both cases the joint could be covered by a strip of aluminum foil set in bituminous paint.

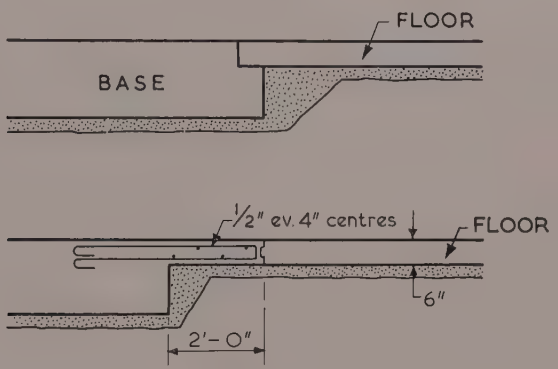


Fig. 9.22.

Stability of Cantilevered Slab Walls

To ensure the stability of a cantilevered slab wall water must be excluded from below the underside of the base. If the edges of the base of the wall are shuttered,

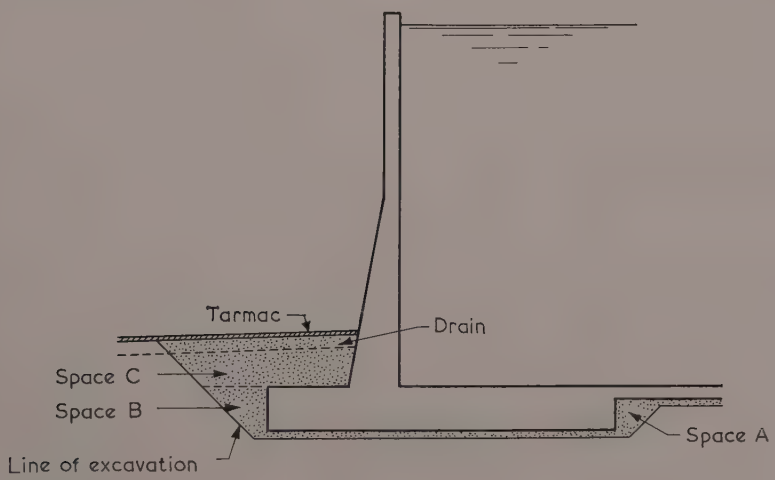


Fig. 9.23.

then spaces A and B in *Fig.* 9.23 must be filled back with reasonably good mass concrete and, in doubtful cases, space C should be filled as well and drains should be provided to lead away rainwater. This is usually possible for service reservoirs

UNIVERSITAS SCIENTIAE

which are normally placed on the highest part of the site. For industrial tanks on low-lying sites where drainage is difficult, walls of this type should not be used.

Net Cost of Wall Base

The tail of the wall in *Fig.* 9.1 also serves as part of the floor. Increasing the width of the tail by 1 ft. saves a strip of floor 1 ft. wide and increases the excavation by only a small amount. Increasing the width of the toe by 1 ft. saves nothing and could appreciably increase the excavation. The combination that gives the minimum combined cost of (wall + floor + excavation) is therefore generally closer to the left-hand side of *Fig.* 9.3 than the dimensions giving the minimum width of base. Instead of a toe-to-tail ratio of 1:2 the best combined answer is generally a ratio of 1:3 to 1:4, the increased width of base being more than offset by savings in the floor and excavation.

Expansion Joints

The spacing of expansion joints in a straight cantilevered slab wall depends on the following factors:

1. The type of cement used.
2. The speed at which the wall is built.
3. The efficiency of the curing during the early life of the concrete.
4. The change of moisture content in the concrete.
5. The range of temperature in the wall.

With slow-hardening cement, rapid construction and efficient curing, the wall of a covered-in reservoir banked round with earth and built in a damp, equable climate does not require expansion joints at less than 100-ft. centres. Some engineers, under such favourable conditions would not provide any joints at all. With the use of a rapid-hardening cement that makes a hard brittle concrete, slow construction causing long time intervals between the concreting of adjoining sections of the wall, and no curing, for the wall of an open reservoir unprotected by banking and built in a country with an extreme range of climatic conditions, a spacing of not more than 30 ft. would be wise.

With average construction in this country spacings of 50 ft. to 100 ft. are usual. The types of joint shown in *Figs.* 16.8 and 22.6 may be used.

CHAPTER TEN

CHANNELS WITH SLOPING BOTTOMS

Long channels of the type shown in *Fig.* 10.1 are sometimes required in sewage works, often in multiple sets of a dozen or so channels side by side. The channels normally run full, but they must be designed for the condition where one channel is full and the adjoining ones empty.

If the sloping bottoms are excavated into really hard ground, the conditions at the base of the vertical wall AB are closely similar to those at the base of a partition wall in a tank with a flat floor, the cantilever moment being shared equally between the floor sections on each side of the wall. If the ground is really soft or

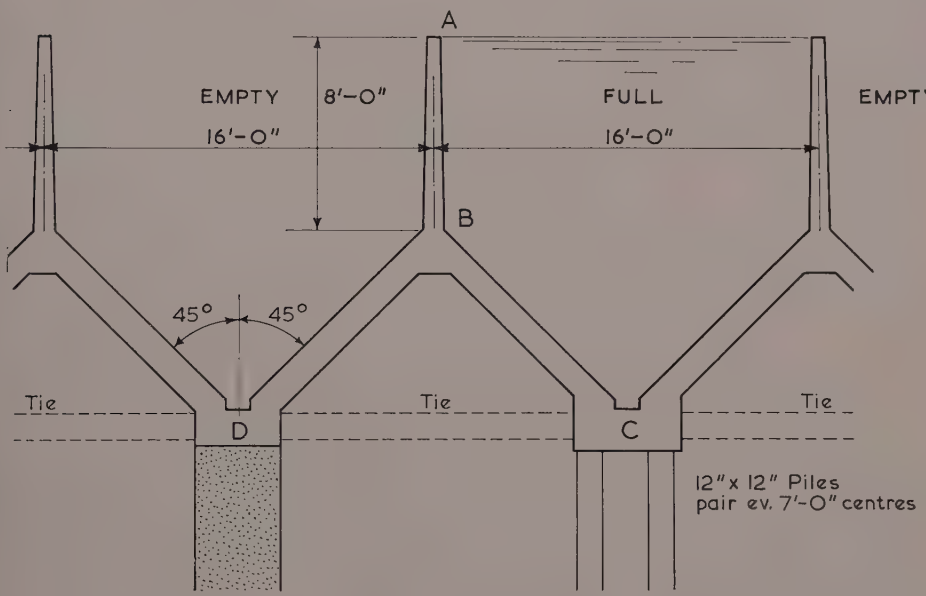


Fig. 10.1.

if the channels are above ground they may be supported at points D and C on mass concrete walls or piles, the horizontal components of the thrusts in the sloping bottoms being taken by ties at intervals.

If the centre-lines of the members in *Fig.* 10.1 are drawn, the height of the vertical wall is about 8·5 ft. and the sloping bottoms are each 11·3 ft. long (*Fig.* 10.2). Assuming the channels hold sewage weighing 65 lb. per cu. ft., the fluid pressure at the base of the wall is $65 \times 8{\cdot}5 = 553$ lb. per sq. ft. The pressure on the sloping bottom varies from 553 lb. per sq. ft. to $65 \times 16{\cdot}5 = 1{,}073$ lb. per sq. ft. The primary fixed-ended moments due to the fluid pressure are

$$M_0 = -\frac{65 \times 8{\cdot}5^3}{6} = -6{,}660 \text{ lb.-ft.}$$

$$M_1 = +\frac{553 \times 11{\cdot}3^2}{12} + (\tfrac{1}{2} \times 520 \times 11{\cdot}3) \times \frac{11{\cdot}3}{15} = +8{,}100 \text{ lb.-ft.}$$

$$M_4 = -\frac{553 \times 11{\cdot}3^2}{12} - (\tfrac{1}{2} \times 520 \times 11{\cdot}3) \times \frac{11{\cdot}3}{10} = -9{,}210 \text{ lb.-ft.}$$

All these moments are per foot run of channel. To the two latter figures moments must be added due to the component of the self-weights of the bottom which, from the figures already worked out, will have to resist a moment of about 10,000 lb.-ft. Working to a resistance moment of $60bd^2$, the bottom will have to be about 14 in. in thickness and the additional moments will be about

$$M_1 = M_3 = +\frac{120 \times 11{\cdot}3^2}{12} = +1{,}280 \text{ lb.-ft.}$$

$$M_2 = M_4 = -\frac{120 \times 11{\cdot}3^2}{12} = -1{,}280 \text{ lb.-ft.}$$

The displacement of the frame may be considered in two stages. Firstly, a rotation of θ_1 of joint B and of θ_2 of joint D; by symmetry, joint C does not rotate. This is shown in *Fig.* 10.2. Secondly, an increase of $2x$ in the distance CD due to the stretching of the tie. This produces a vertical fall at point B, the deflection of B below C and D being Δ where $\Delta = x/\sin\alpha$, as shown in *Fig.* 10.3. Tabulating the values of the moments:

	M_1	M_2	M_3	M_4
All joints fixed	+9,380	−1,280	+1,280	−10,490
Due to θ_1	$4EK\theta_1$	$4EK\theta_1$	$2EK\theta_1$	$2EK\theta_1$
,, ,, θ_2	—	$2EK\theta_2$	$4EK\theta_2$	—
,, ,, Δ	$-6EK\frac{\Delta}{L}$	$6EK\frac{\Delta}{L}$	$6EK\frac{\Delta}{L}$	$-6EK\frac{\Delta}{L}$

$M_0 + M_1 + M_2 = 0.$

$$-6{,}660 + 9{,}380 + 4EK\theta_1 - 6EK\frac{\Delta}{L} - 1{,}280 + 4EK\theta_1 + 2EK\theta_2 + 6EK\frac{\Delta}{L} = 0.$$

$$8EK\theta_1 + 2EK\theta_2 = -1{,}440.$$

The value of θ_2 may be very small if the support is a continuous wall. Taking a normal continuous die-away coefficient it may be assumed that $\theta_2 = -0{\cdot}33\theta_1$, giving

$$EK\theta_1 = -\frac{1{,}440}{7{\cdot}33} = -197.$$

It may appear that Δ is primarily an unknown but if it is decided to provide a tie working to a definite stress a definite value for Δ is known. If a tie working to exactly 200 lb. per sq. in. is provided and E is taken as 3,000,000 lb. per sq. in.,

$$2x = \frac{200}{3{,}000{,}000} \times 16 \text{ ft.}$$

$$\Delta = \frac{x}{\sin\alpha} = \frac{100}{3{,}000{,}000} \times \frac{16}{\sin 45 \text{ deg.}} = \frac{754}{1{,}000{,}000} \text{ ft.}$$

$K = \frac{I}{L} = \frac{T^3}{12} \times \frac{1}{L}$, where T is the thickness of the sloping floor and $L = 11{\cdot}3$ ft. Taking T as 1·17 ft.,

$$K = \frac{1{\cdot}17^3}{12} \times \frac{1}{11{\cdot}3} \times 0{\cdot}0118 \text{ foot units.}$$

$6EK\frac{\Delta}{L}$ in foot units $= 6 \times (3{,}000{,}000 \times 144) \times 0{\cdot}0118 \times \frac{754}{1{,}000{,}000} \times \frac{1}{11{\cdot}3}$

$= 2{,}040$ lb.-ft.

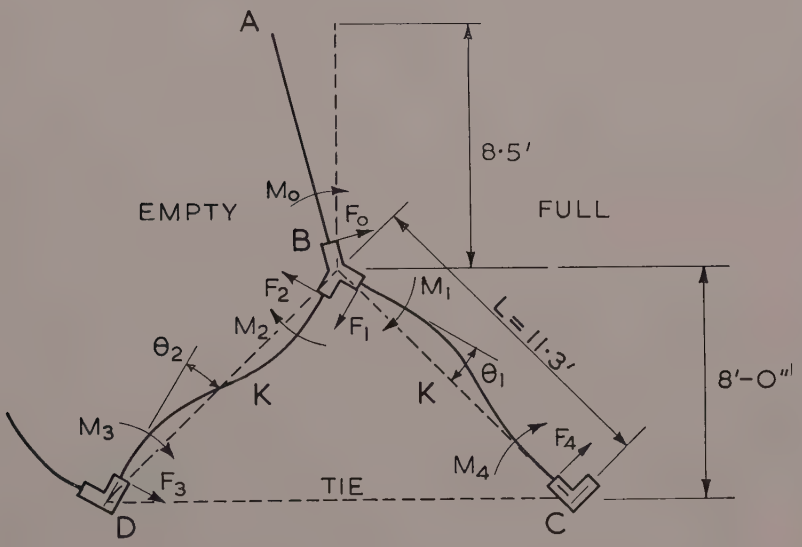


Fig. 10.2.

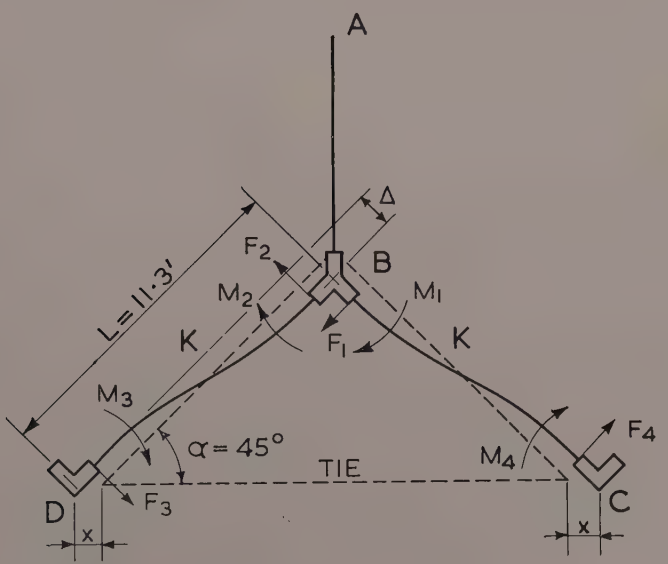


Fig. 10.3.

Final $M_1 = +9{,}380 + 4EK\theta_1 - 6EK\frac{\Delta}{L}$

$= +9{,}380 - 790 - 2{,}040 = +6{,}550$ lb.-ft.

Final $M_2 = -1{,}280 + 3{\cdot}33EK\theta_1 + 6EK\frac{\Delta}{L}$

$= -1{,}280 - 656 + 2{,}040 = +104$ lb.-ft.

Final $M_3 = +1{,}280 + 0{\cdot}67EK\theta_1 + 6EK\frac{\Delta}{L}$

$= +1{,}280 - 132 + 2{,}040 = +3{,}188$ lb.-ft.

Final $M_4 = -10{,}290 + 2\mathrm{EK}\theta_1 - 6EK\frac{\Delta}{L}$

$= -10{,}490 - 394 - 2{,}040 = -12{,}924$ lb.-ft.

Tabulating the shears,

	F_1	F_2	F_3	F_4
All fixed	$+4{,}678$	-678	$+678$	$-5{,}858$
Due to θ_1	$6EK\frac{\theta_1}{L}$	$6EK\frac{\theta_1}{L}$	$6EK\frac{\theta_1}{L}$	$6EK\frac{\theta_1}{L}$
„ „ θ_2	—	$6EK\frac{\theta_2}{L}$	$6EK\frac{\theta_2}{L}$	—
„ „ Δ	$-12EK\frac{\Delta}{L^2}$	$12EK\frac{\Delta}{L^2}$	$12EK\frac{\Delta}{L^2}$	$-12EK\frac{\Delta}{L^2}$

$F_0 = -\frac{65}{2} \times 6{\cdot}5^2 = -1{,}375$ lb.

The primary fixed-ended value of

$$F_1 = +\left(553 \times \frac{11{\cdot}3}{2}\right) + \frac{3}{10}\left(\frac{1}{2} \times 520 \times 11{\cdot}3\right) + \left(120 \times \frac{11{\cdot}3}{2}\right)$$

$= 3{,}120 + 880 + 678 = 4{,}678$ lb.

$F_2 = -678$ lb. $\quad F_3 = +678$ lb.

$F_4 = -3{,}120 - 2{,}060 - 678 = -5{,}858$ lb.

$$6EK\frac{\theta_1}{L} = 6 \times \left(-\frac{197}{11{\cdot}3}\right) = -104 \text{ lb.}$$

$$12EK\frac{\Delta}{L^2} = \frac{2}{L} \times 6EK\frac{\Delta}{L} = \frac{2}{11{\cdot}3} \times 2{,}040 = 361 \text{ lb.}$$

The final shears are as follows:

$F_1 = 4{,}678 - 104 - 361 = +4{,}213$ lb.
$F_2 = -678 - 69 + 361 = -386$ lb.
$F_3 = +678 - 69 + 361 = +970$ lb.
$F_4 = -5{,}858 - 104 - 361 = -6{,}323$ lb.

The direct thrusts and shears on joint B are shown in *Fig.* 10.4.

T_0 = weight of wall, say $8{\cdot}5 \times 100 = 850$ lb.
Resolving vertically:

$$T_1 \cos 45 \text{ deg.} + T_2 \cos 45 \text{ deg.} = T_0 + F_1 \cos 45 \text{ deg.} - F_2 \cos 45 \text{ deg.}$$
$$T_1 + T_2 = 5{,}799 \text{ lb.}$$

Resolving horizontally:

$$T_2 \cos 45 \text{ deg.} - T_1 \cos 45 \text{ deg.} = -F_0 + F_2 \cos 45 \text{ deg.} + F_1 \cos 45 \text{ deg.}$$
$$T_2 - T_1 = 1{,}882 \text{ lb.}\quad T_1 = 1{,}959 \text{ lb. and } T_2 = 3{,}840 \text{ lb.}$$

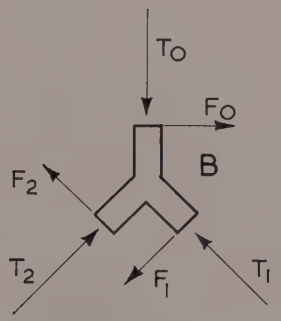


Fig. 10.4.

The total vertical reaction on the support at C is $2T_1 \cos 45 \text{ deg.} - 2F_4 \cos 45 \text{ deg.}$
$= 2{,}750 + 8{,}950 = 11{,}700$ lb.
When all the channels are full this increases to 17,690 lb. per foot run of channel.
The tension in the tie may be taken as $T_2/\sqrt{2}$, or $\frac{3{,}840}{1{\cdot}414} = 2{,}720$ lb. per foot run of channel.

If the ties are placed at 7-ft. centres, each takes $7 \times 2{,}720 = 19{,}000$ lb. At 200 lb. per sq. in., a sectional area of 95 sq. in. is required, or say, 7 in. by 14 in.
Free moment on span BC is very nearly

$$(553 + 120) \times \frac{11{\cdot}3^2}{8} = 10{,}720 \text{ lb.-ft.}$$
$$+ 2{,}940 \times \frac{11{\cdot}3}{8} = 4{,}160 \text{ „ „}$$
$$14{,}880 \text{ lb.-ft.}$$

The moments in AB, BC and BD are as shown in *Fig.* 10.5. These moments are based on the assumption that there will be rotation of joint D and that stretching of the ties occurs.

If the ground turns out to be harder than has been assumed, there may be very little rotation at D and very little tension in the ties. The case where rotation and tension occurs has been dealt with. The moments under the three further conditions must also be checked.

2. Rotation at D but with no tension in the tie (retain θ_2, but write $\Delta = 0$).
3. No rotation at D but with tension in the tie (retain Δ, but write $\theta_2 = 0$).
4. No rotation at D and no tension in the tie (write $\theta_2 = 0$ and $\Delta = 0$).

In cases such as this the passive resistance of the earth may take up all the

horizontal component of T_2. In the foregoing example the effect of the stretching of the tie appears as a separate item in the calculation of the bending moments as $\pm 6EK\frac{\Delta}{L}$. With a tensile stress of 200 lb. per sq. in. in the ties, the induced moments are $\pm$2,040 lb.-ft. If the tensile stress in the tie is restricted to 100 lb. per sq. in.,

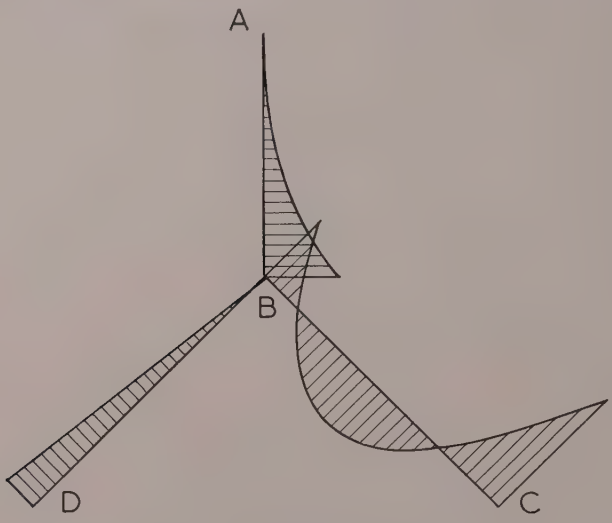


Fig. 10.5.

the induced moments fall to $\pm$1,020 lb.-ft. If the passive earth pressure takes all unbalanced horizontal thrust they fall to zero. This would give

$$M_1 = +8{,}590 \text{ lb.-ft.} \quad M_2 = -1{,}936 \text{ lb.-ft.}$$
$$M_3 = +1{,}148 \text{ lb.-ft.} \quad M_4 = -10{,}884 \text{ lb.-ft.}$$

The reader is left to draw the moment diagrams for the three other combinations of conditions.

If the channel is supported at intervals it may be carried on a beam (each channel when full of water weighs about 8 tons per foot), or the two sloping sides may be made to carry 0·707 times the total weight, but in this case the stresses due to this longitudinal bending must be added to those caused by the cross bending, which have already been calculated. If each sloping side acts as a girder 14 in. thick and 150 in. deep overall and carries 12,700 lb. per ft. over a span of 7 ft., the reversed moment is

$$12{,}700 \times 7^2 \times \frac{12}{12} = 623{,}000 \text{ lb.-in.}$$

The stresses in the concrete are about $\pm \dfrac{623{,}000 \times 6}{14 \times 150^2} = \pm 12$ lb. per sq. in. only. The two channels on the extreme flanks of the set of channels will require special treatment such as carrying the vertical wall down to a separate line of foundations.

CHAPTER ELEVEN

CANTILEVERED COUNTERFORT WALLS

The complications introduced by the use of counterforts are of the type that cannot usually be divided into a number of simple independent problems to be solved separately. It may be necessary to prepare a preliminary design of the wall as a whole and to study this before the designer can see exactly what problems are involved and how best they may be solved. This method is followed later in this chapter. First, however, a design will be commenced which eventually turns out to be unworkable, as this best illustrates some of the difficulties which may be encountered. Only by preparing a number of preliminary designs and estimates can the best solution be found for any particular case.

General Design

Many early counterforts were of the type in *Fig.* 11.1*a*, where the wall slab and base slab both span from counterfort to counterfort. When a projecting toe

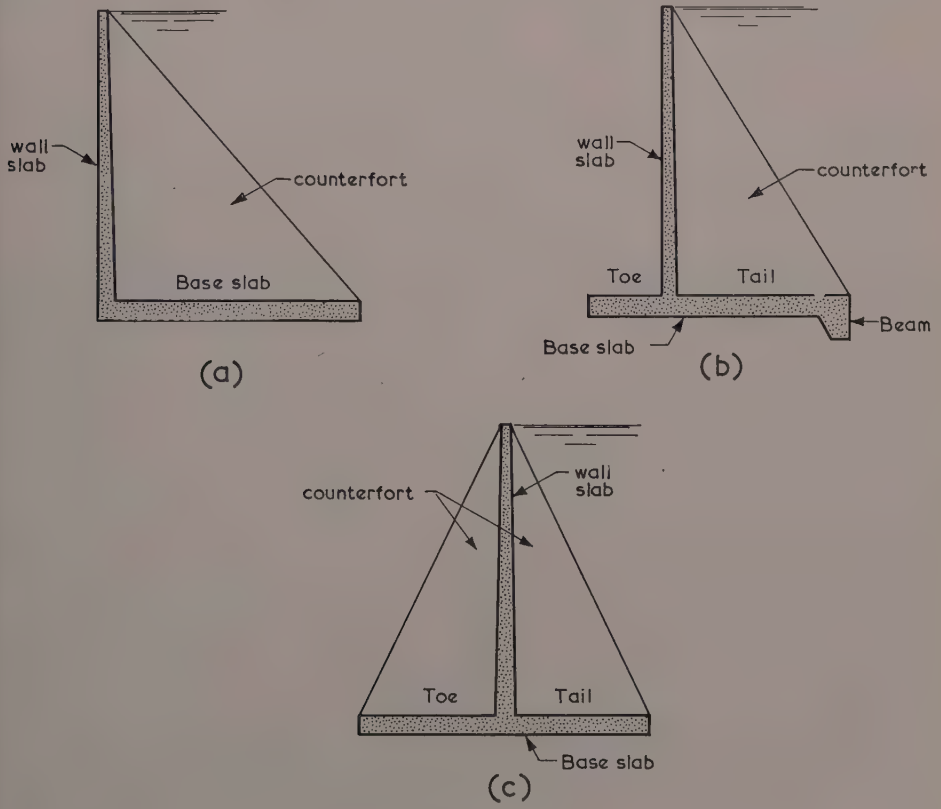


Fig. 11.1.

was required to reduce the bearing pressure on the ground or to stabilize the wall against earth pressure this arrangement was modified by adding a beam at the back of the tail (*Fig.* 11.1*b*), the base slab then spanning from beam to wall instead of from counterfort to counterfort. Occasionally a second counterfort was added in front of the wall, as in *Fig.* 11.1*c*, with the base slab on both sides of the wall spanning from counterfort to counterfort, but this arrangement was unusual except for division walls. The reinforcement in the counterfort was designed for a stress of 16,000 lb. per sq. in. and no attempt was made to calculate the tensile stresses in the concrete due to bending moments. If the recommendations given in CP.2007:1960 are to be taken seriously then the stress in the steel in an internal counterfort must be kept below 12,000 lb. per sq. in. and the tensile stress in the concrete below about 300 lb. per sq. in. Provdide that the bars are well anchored there can be no question of structural failure and there is no question of leakage through the counterfort. The sole reason for limiting the tensile stress in the concrete must therefore be to prevent cracking, which might lead to rusting of the

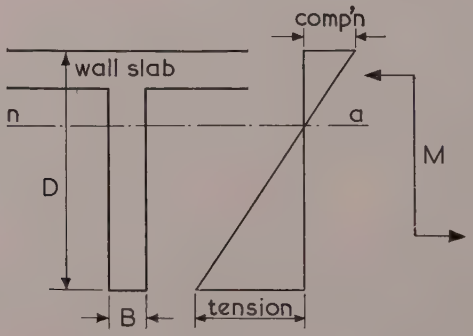


Fig. 11.2.

steel. In the writer's experience such deterioration is due firstly to poor, overcrowded bar arrangements, making it impossible for the concrete to penetrate between the bars; secondly to the use of harsh, dry mixes leading to honeycombing on the ' blind ' side of the bars; and thirdly, and most frequently, to the failure to specify and provide proper bar spacers to maintain the specified cover. The writer has some sympathy with the reduction of the working stress in the steel from 16,000 to 13,500 lb. per sq. in., provided the laps are not shortened pro-rata, but if there is no evidence of deterioration of internal counterforts due to high calculated tensile stresses in the concrete there seems no point in taking precautions against a risk that does not exist.

A horizontal plan-section through a counterfort, subjected to a moment M is shown in *Fig.* 11.2, with the position of the neutral axis close to the wall slab. Ignoring the reinforcement, a moment applied to this section will produce a tensile stress in the counterfort some two or three times as large as the compression stress in the wall slab.

Accepting the outlook adopted in CP.2007:1960, that the tensile stress in the concrete should not exceed 300 lb. per sq. in., although the compressive stress may be 1,200 lb. per sq. in. a natural arrangement from the theoretical point of view is to use a counterfort such as that shown in *Fig.* 11.3, with the wall slab in tension and the counterfort in compression. The most frequent location of leaks is through

the horizontal construction joints in the wall slab and this type of counterfort causes tension across these joints. The construction of the counterfort below the base

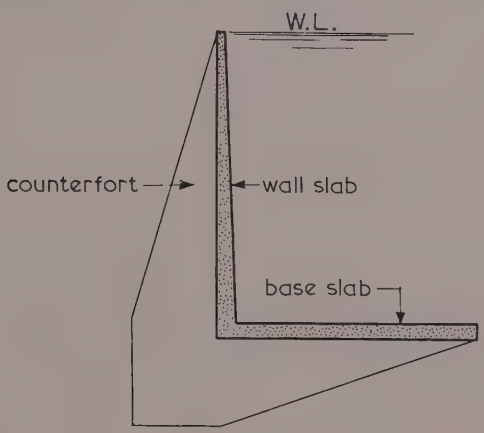


Fig. 11.3.

slab, although not as difficult as it may appear at first sight, is sufficiently complicated to rule out this form of construction in most cases.

Counterforts of special shapes are discussed later.

Wall Slab

Whatever the shape or spacing of the counterforts it is standard practice to span the wall slab from counterfort to counterfort without introducing any horizontal beams. The use of segmental arches or slabs having splayed sections is discussed in Chapters 14 and 15. In the following a straight slab without splays spanning horizontally from counterfort to counterfort is assumed.

The thickness of the wall slab normally increases from top to bottom and the minimum thickness at any depth is found as follows. If h (feet) is the depth below the surface at any point and L (feet) is the spacing of the counterforts, the reversed moment is $\frac{1}{12}whL^2$ lb.-ft. per foot height where w (pounds) is the weight per cu. ft. of liquid inside the reservoir.

Working to a resistance moment of $60bd^2$ lb.-in. and taking w as 62·5 lb. per cu. ft., $60 \times 12 \times d^2 = \frac{1}{12}whL^2$ lb.-in. Minimum $d = 0{\cdot}295L\sqrt{h}$ inches.

Since L does not vary from top to bottom of any given wall, the minimum value of d varies as $\sqrt{h}$. This is a convex curve and cannot always be covered by a straight line drawn from a thickness of 6 in. at the top to the maximum value at the base.

For example if $L = 15$ ft. for a wall 30 ft. in height, d at a depth of 10 ft. $= 0{\cdot}295\sqrt{10} = 13{\cdot}98$ in. At depths of 5 ft., 15 ft., 20 ft., 25 ft. and 30 ft., the minimum values for d are 9·86 in., 17·1 in., 19·8 in., 22·1 in. and 24·2 in. Plotting these (to unequal scales) in *Fig.* 11.4, it will be seen that a uniform taper from 9 in. to 25 in. will provide the required thickness at all depths whereas a uniform taper from 6 in. to 25 in. will not. If, therefore, a slab is required to span 15 ft. for a wall 30 ft. in height and designed for a resistance moment of $60bd^2$, it must be 9 in. in thickness at a point 6 in. above water level and 25 in. at the base (see *Fig.* 11.4). (This neglects secondary effects which are discussed later.)

The thickness thus increases 16 in. in a height of 30 ft. 6 in. At a depth of 15 ft. below water level the thickness of the wall is 17·14 in.

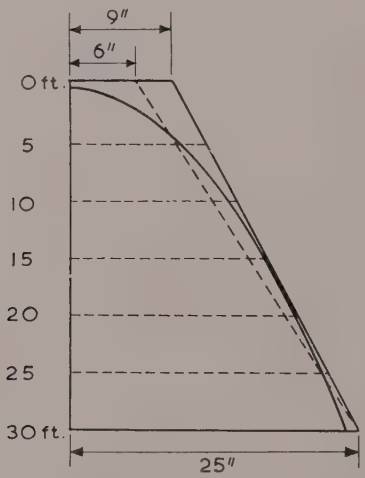


Fig. 11.4.

Reversed $M = \frac{1}{12} \times 15 \times 62{\cdot}5 \times 15^2 \times 12 = 211{,}000$ lb.-in. per foot of height.

Minimum $d = \sqrt{\frac{211{,}000}{60 \times 12}} = 17{\cdot}14$ in.

Taking the effective depth d_1 as 15·14 in.,

$$A_{st} = \frac{211{,}000}{12{,}000 \times 0{\cdot}865 \times 15{\cdot}14} = 1{\cdot}34 \text{ sq. in.}$$

The moment at midspan is only $\frac{1}{24} wL^2$ and at this point the steel stress may be 18,000 lb. per sq. in., so that the amount of steel required to resist the positive moment will be about 0·33 times that for the reversed moment, or 0·45 sq. in.

If reinforcement is provided equal to 0·15 per cent. of the section, the area required is

$$\frac{0{\cdot}15}{100} \times 12 \times 17{\cdot}14 = 0{\cdot}31 \text{ sq. in.}$$

If the wall is later to be banked with earth which exerts a pressure of 20 lb. per sq. ft. per foot of depth, the reversed moment at a depth of 15 ft. will be

$$\frac{1}{12} \times 15 \times 20 \times 15^2 \times 12 = 67{,}500 \text{ lb.-in.}$$

$$A_{st} = \frac{67{,}500}{18{,}000 \times 0{\cdot}9 \times 15{\cdot}14} = 0{\cdot}276 \text{ sq. in.}$$

Since the moment due to earth pressure does not function unless the tank is empty, the area of steel required to resist the positive earth moment will be one-half of this amount, or 0·138 sq. in. All these values are shown in the upper part of *Fig.* 11.5.

Calculating areas of main reinforcement required at depths of 5 ft., 10 ft., 20 ft., 25 ft. and 30 ft., the values shown in *Fig.* 11.6 are obtained. A proposed arrangement for the steel in the wall slab is shown in the lower part of *Fig.* 11.5 and at the right-hand side of *Fig.* 11.6. The longer bars *b* lap at each counterfort. Between

are two short bars *a*, giving four bars in three spacings at the counterforts. The spacings are shown in *Fig.* 11.6. At the bottom of the wall the spacing is 4 in. or four $\frac{3}{4}$-in. diam. bars in three spacings of 4 in. = 1·76 sq. in. per foot of height. At a depth of 15 ft. the spacing is 5 in. or four $\frac{3}{4}$-in. diam. bars in 15 in., equal to

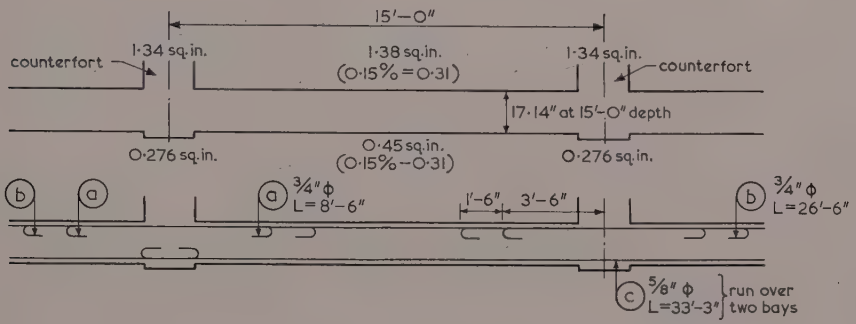


Fig. 11.5.

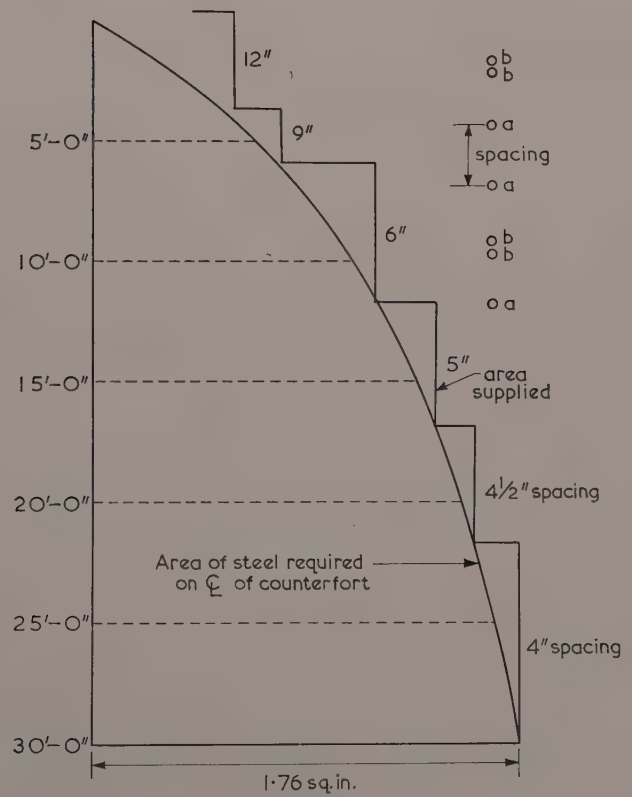


Fig. 11.6.

1·41 sq. in. per foot of height near the water face at the counterfort. Halfway between the counterforts near the water face, one bar *b* is provided per 15 in. or 0·353 sq. in. per foot of height. The bars near the earth face are $\frac{5}{8}$-in. diam. bars at the same 'spacing' as the bars near the water face, that is $\frac{5}{8}$-in. diam. bars at

5-in. centres, or 0·74 sq. in. per foot of height. In a short wall, well protected from changes of temperature these bars could be $\frac{1}{2}$-in. in diameter.

The vertical distribution steel may be arranged in two flights of about 16 ft. 6 in. The bottom half of the wall varies in thickness from 17 in. to 25 in., the average being 21 in. The net width of slab between counterforts is about 13 ft. 3 in. Taking 0·3 per cent. of 21 in. by 13 ft. 3 in. gives 10 sq. in. Using twenty-two $\frac{3}{4}$-in. diam. bars, the area provided is 9·7 sq. in. The average thickness of the top half of the wall is 13 in. Taking 0·3 per cent. of 13 in. by 13 ft. 3 in. gives 6.2 sq. in. Using twenty-two $\frac{5}{8}$-in. diam. bars gives 6·74 sq. in.

From *Figs.* 11.5 and 11.6 the weight of reinforcement in one 15-ft. wide panel of wall slab can be determined. This consists of 46 bars of type *a*, 23 bars of type *b*, 69 half-bars *c*, twenty-two $\frac{3}{4}$-in. diam. vertical bars 16 ft. 6 in. in length, twenty-two $\frac{5}{8}$-in. diam. vertical bars, 16 ft. 3 in. long, and twenty-two $\frac{1}{2}$-in. diam. hairpins at the top of the wall 5 ft. in length, giving a total of 33·3 cwt. Taking the counterforts as 21 in. in thickness (see later) the concrete in one net panel is 13 ft. 3 in. wide, 30 ft. 6 in. high and 9 in. to 25 in. in thickness, or 21·2 cu. yd. per 15-ft. wide panel.

The load carried by each counterfort varies directly as the spacing. If the spacing of the counterforts is doubled then they must be made twice as thick and will need twice as much reinforcement, so that the cost of the concrete and steel in the counterforts per foot run of wall is constant and independent of their spacing. This also applies to the shuttering up the front and back edges of the counterforts. The only variable factor, as far as the counterforts themselves are concerned, is the shuttering to their cheeks. With a triangular counterfort 18 ft. wide at the base, this amounts to 0·5 × 18 × 30·5, or, say, 30 sq. yd. per face. If the spacing is reduced from 15 ft. to 7 ft. 6 in., then each counterfort will be only half as thick and contain half as much steel, but there will be two counterforts per 15 ft. of wall and an increase of 60 sq. yd. of shuttering.

Repeating the calculations for spacings of 10 ft. and 7 ft. 6 in., the quantities can be calculated per 15-ft. run of wall.

Assume that the relative unit costs are are follows:

Concrete per cu. yd.: 4 units
Mild steel rods per cwt.: 3 units
Shuttering per sq. yd.: 1 unit
Hand excavation per cu. yd.: 1 unit.

The relative costs of the counterforts and wall slab per 15-ft. run of wall (including only the variable factors) are then as follows:

15-ft. Spacing—	Concrete: 21·2 cu. yd. × 4	=	84·8
	Steel: 33·3 cwt. × 3	=	99·9
	Shuttering: 60 sq. yd. × 1	=	60·0
	Total:		244·7
10-ft. Spacing—	Concrete: 15 cu. yd. × 4	=	60·0
	Steel: 22·6 cwt. ×3	=	67·8
	Shuttering: 90 sq. yd. × 1	=	90·0
	Total	=	217·8
7-ft. 6-in. Spacing—	Concrete: 11·2 cu. yd. × 4	=	44·8
	Steel: 19·4 cwt. × 3	=	58·2
	Shuttering: 120 sq. yd. × 1	=	120·0
	Total	=	223·0

These figures do not represent the whole story, as the base slab and the back beam (if any) have not been included. Assuming that the base slab is designed to span from wall to back beam in all cases, then its cost per foot run of wall is constant. The cost of the back beam increases with the spacing of the counterforts, and if this is added to the foregoing figures, the 15-ft. counterfort spacing will be the most expensive and the 7-ft. 6-in. spacing the cheapest.

These figures are based on certain assumed working stresses, certain assumed unit costs and a certain size of counterfort. Varying the working stresses, the relative unit costs or the size of the counterfort will produce a different answer. The reader must be prepared to run out and price alternative schemes.

General Proportions

The total width of base must be similar to that needed for a cantilevered slab wall. The correct proportion of the base that should project in front of the face of the wall to form a toe and the correct spacing of the counterforts are problems that may be complicated by secondary effects.

Instead of devoting time and space to a general discussion of these matters, a particular wall with a simple triangular counterfort will be considered, suitable proportions and spacings assumed, a preliminary design completed (ignoring secondary effects), and any modifications necessary will then be discussed.

Before becoming involved in the arithmetical calculation for any actual wall a close approximate picture of the actions of the primary loads and moments can be obtained by studying a thin, weightless, theoretical wall of similar proportions. For a cantilevered-slab wall the minimum width of base is 0·863 times the height, one-third of this base projecting in front of the wall face to form a toe (see *Figs.* 9.2 and 9.3). Taking these same proportions for a counterfort wall, the theoretical wall shown on the left-hand side of *Fig.* 11.7 is obtained. With a thin weightless wall of these proportions the resultant passes exactly through the middle-third point of the base and the maximum pressure on the ground is 1·33 wH. The base slab cantilevers a distance of 0·288H in front of the wall and then spans 0·575H from the wall to the back beam, being freely supported by a downward reaction R_F supplied by the wall slab and an upward reaction R_B supplied by the back beam, which spans from counterfort to counterfort.

A wall 30 ft. in height with counterforts spaced at 15-ft. centres and with the general proportions shown in *Fig.* 11.7 will be considered. As with the cantilevered slab wall the effective height of a practical wall is measured from water level to the underside of the base slab and not the net depth of water (see *Fig.* 9.1). The moment in the base slab is $0{\cdot}052wH^3$ (*Fig.* 11.7). If $H = 30$ ft., $M = 0{\cdot}052 \times 62{\cdot}5 \times 30^3$ or 87,700 lb.-ft. per foot run. This causes tension on the underside of the slab. With a resistance moment of $250bd_1^2$, $d_1 = \sqrt{\dfrac{87{,}700 \times 12}{12 \times 250}} = 18{\cdot}75$ in.

The thickness of the base slab will therefore be about 2 ft. and the effective height of the wall about 32 ft.

Overall width of base = 0·863H, say 0·863 × 32 or 27·6 ft. For a preliminary design the base can be made 27 ft. wide, with 9 ft. projecting in front of the centre-line of the wall and 18 ft. behind (see *Fig.* 11.8).

The section modulus of a T-beam of average proportions as shown in *Fig.* 11.2 is about $0{\cdot}25BD^2$ for tensile stresses. For the theoretical wall in *Fig.* 11.7 the value

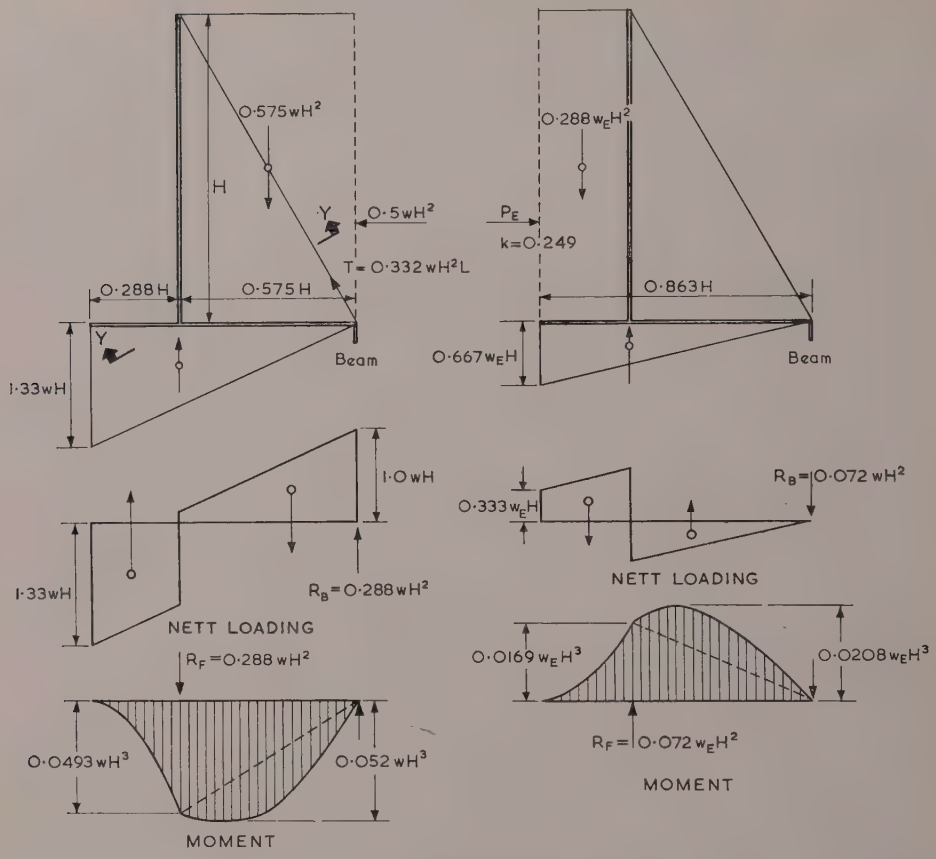


Fig. 11.7.

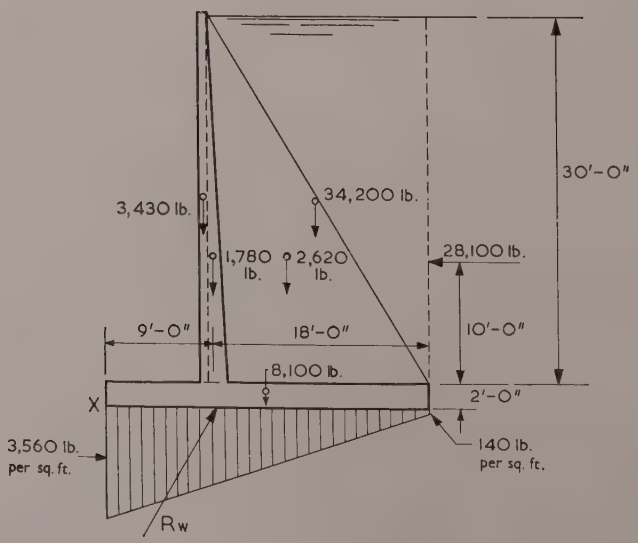


Fig. 11.8.

of D for Section Y–Y is $0{\cdot}498H$ The moment on Section Y–Y is $0{\cdot}167wH^3L$, where L is the spacing of the counterforts.

If this T-section is stressed to 300 × 144 lb. per sq. ft.,

$$0{\cdot}167wH^3L = \text{say } 0{\cdot}25B \times (0{\cdot}498H)^2 \times 300 \times 144 \text{ or } \frac{B}{L} = \frac{wH}{1{,}610}.$$

If $w = 62{\cdot}5$ and $H = 30$, $B = 0{\cdot}117L$.

If $L = 15$ ft., then $B = 0{\cdot}117 \times 15 = 1{\cdot}75$ ft. or 21 in.

All these values are preliminary theoretical figures and will be modified in practice by the thickness and weight of the members but they are sufficiently accurate for a preliminary design.

A design has already been prepared for the wall slab, which varies from 9 in. at the top to 25 in. at the bottom (*Figs.* 11.4, 11.5 and 11.6).

In the right-hand half of *Fig.* 11.7 this theoretical wall is shown subjected to an external earth pressure when empty and here a difficulty arises. In a normal earth-retaining wall the intensity of pressure on the foundation is greatest on the side further from the earth and the wall tends to move away from the earth pressure. With a wall of the proportions shown in *Fig.* 11.7 the greatest pressure on the foundation is on the same side as the earth, the wall leaning towards the earth pressure and thus tending to turn the normal small active earth pressure into a larger passive counter-pressure. The extent to which this develops depends on the relative vertical compressibility of the foundation (the factor C_2) and the horizontal compressibility of the filling. Little is as yet known about the former and practically nothing about the latter, so the answer is largely a matter of guesswork.

In *Fig.* 11.7 the earth pressure has been assumed to be just sufficient to keep the resultant on the middle-third point, since this is the condition of the wall when the reservoir is full and no banking has been placed. The banking would, most likely, be placed when the reservoir is full as that is the normal condition of most reservoirs.

To produce this result with a wall of these proportions a lateral pressure of $0{\cdot}249w_E$ is necessary, where w_E is the unit weight of the earth banking. If $w_E = 100$ lb. per cu. ft., the pressure is 24·9 lb. per sq. ft. per foot of depth.

Again assuming that a back beam is provided and that all the loading on the base slab is carried by a slab spanning front-to-back, the loading, moments and reactions shown in the lower right-hand part of *Fig.* 11.7 are obtained.

The value of k of 0·249 is merely an abstract figure to stabilize the theoretical wall. The weight and thickness of the members of an actual wall will modify this value.

Stability of Actual Wall

In *Fig.* 11.8 the loading on the actual wall (per foot run) when subject to water pressure is shown. In this figure 34,200 lb. is the gross weight of the water and 1,780 and 2,620 lb. are the submerged weights of the concrete. Counterforts 1 ft. 9 in. in thickness at 15-ft. centres have been allowed for and the small weight of the back beam has been omitted.

Taking moments about point X, the loading on the ground is as shown in *Fig.* 11.8. The shear on the ground immediately below the base is $\frac{28{,}100}{27} = 1{,}040$ lb. per sq. ft.

The net loading on the base slab is as shown in *Fig.* 11.9. Allowing for the self-weight of the base slab and the weight of water standing over the tail the loading varies from (3,560 − 300) to {140 − [300 + (30 × 62·5)]} or from 3,260 to −2,035 lb. per sq. ft. The slab is held down by a reaction R_F and is supported by a reaction R_B from the back beam. Assuming the width of this beam is 2 ft., the slab cantilevers 9 ft., spans 17 ft. and then cantilevers 1 ft.

Taking moments about R_F gives R_B = 19,100 lb.

The moment on the 9-ft. cantilever is 24,200 × 4·82 = 116,500 lb.-ft.

The load on the 1-ft. cantilever is about 2,000 lb., and the moment is about 2,000 × 0·5 = 1,000 lb.-ft.

The load standing over the 17-ft. span is 14,400 lb., and the free moment caused by this load is about $\frac{1}{8}$ × 14,400 × 17 = 30,600 lb.-ft.

Drawing the broken line from the moment of 116,500 lb.-ft. at R_F (tension on underside) to the moment of 1,000 lb.-ft. at R_B (tension on upper side) and drawing a parabola 30,600 lb.-ft. high on this line the complete bending-moment diagram for the base slab is obtained as shown in *Fig.* 11.9. This diagram is sufficiently

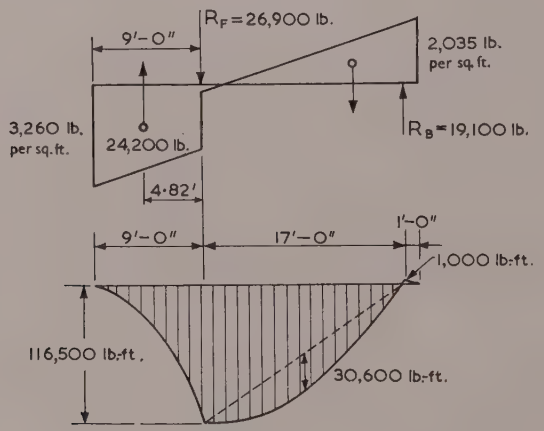


Fig. 11.9.

accurate for a preliminary design. For a final design the intermediate moments must be calculated.

Assuming that the reservoir is banked round with earth after being filled and tested, and assuming that the earth weighs 100 lb. per cu. ft. and that it exerts a pressure of 20 lb. per sq. ft. per foot of depth, the loading per foot run of wall when the reservoir is empty is as shown in *Fig.* 11.10. When the self-weight of the wall is included the resultant passes inside the middle-third point and there is no need for any counter-pressure to develop. Taking moments about point X the pressures on the ground may be obtained. These give the net loading on the base slab shown in the upper part of *Fig.* 11.11 and the moment diagram shown in the lower part of the same figure.

Base Slab

The maximum moment is 116,500 lb.-ft. (see *Fig.* 11.9) causing tension on the underside. With a resistance moment of $250bd_1^2$.

$d_1 = \sqrt{\dfrac{116{,}500 \times 12}{250 \times 12}} = 21{\cdot}6$ in.; say 24 in. deep overall (as already estimated).

$A_{st} = \dfrac{116{,}500 \times 12}{18{,}000 \times 0{\cdot}865 \times 22} = 4{\cdot}07$ sq. in.; say $1\frac{1}{4}$-in. diam. bars at $3\frac{1}{2}$-in. centres, (4·21 sq. in.).

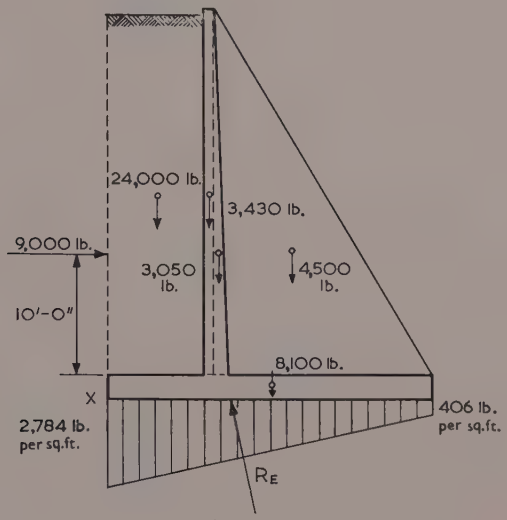
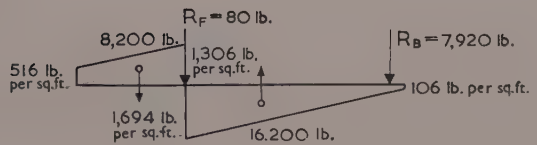


Fig. 11.10.



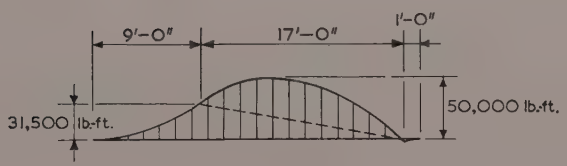


Fig. 11.11.

The maximum moment causing tension on the water face is 50,000 lb.-ft. (see *Fig.* 11.11). If it is assumed that the reservoir, when empty, is not a ' water-containing ' structure no difficulty arises. If, however, this slab has to be designed for a resistance-moment value of $60bd^2$ then minimum $d = \sqrt{\dfrac{50{,}000 \times 12}{60 \times 12}} = 28{\cdot}9$ in., and the thickness would have to be increased. Taking the view that this is not necessary $A_{st} = \dfrac{50{,}000 \times 12}{18{,}000 \times 0{\cdot}9 \times 22} = 1{\cdot}69$ sq. in.; say $1\frac{1}{8}$-in. diam. bars at 7-in.

centres (1·70 sq. in.). This is cutting the design rather fine and in the detail in *Fig.* 11.12, 1¼-in. diam. bars at 7-in. centres are shown.

If 0·15 per cent. of distribution bars are provided, the area required is $\frac{0{\cdot}15}{100} \times 12 \times 24 = 0{\cdot}432$ sq. in.; say ¾-in. bars at 12-in. centres near both faces. However much steel is provided in the front-to-back direction, the base slab must also tend to span from counterfort to counterfort and in the detail in *Fig.* 11.12

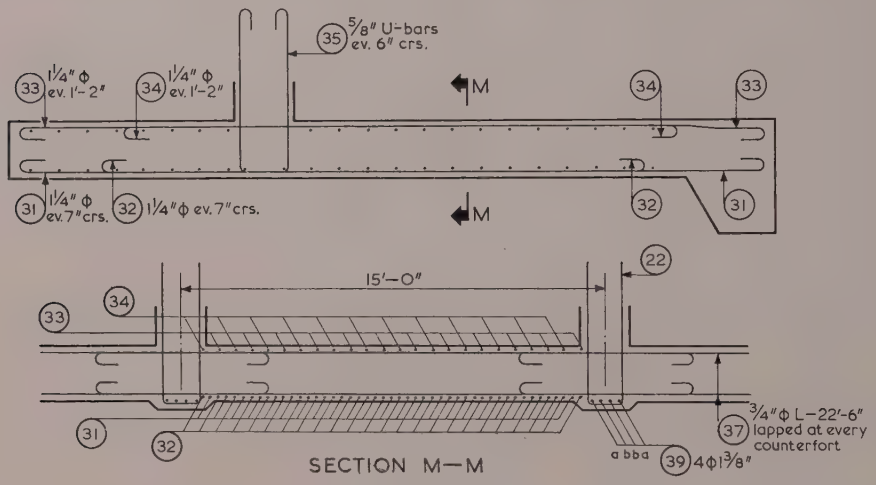


Fig. 11.12.

the distributors are lapped so as to double the area of steel under each counterfort.

Assuming an effective value of H of 32 ft., the moments in *Figs.* 11.9 and 11.11 can be checked against the values in *Fig.* 11.7.

$0{\cdot}052wH^3 = 0{\cdot}052 \times 62{\cdot}5 \times 32^3 = 106{,}800$ lb.-ft., against 116,500 lb.-ft.
$0{\cdot}0208w_EH^3 = 0{\cdot}0208 \times 100 \times 32^3 = 68{,}200$ lb.-ft.

But this is for a value of $k = 0{\cdot}249$. For $k = 0{\cdot}20$ the expected value would be about 54,800 lb.-ft., as against 50,000 lb.-ft.

The suspension reaction of $0{\cdot}072w_EH^2$ in *Fig.* 11.7 is $0{\cdot}072 \times 100 \times 32^2 = 7{,}400$ lb. ($k = 0{\cdot}249$). In *Fig.* 11.11 this has turned into a downward reaction of 80 lb. (when $k = 0{\cdot}20$).

Back Beam

The load carried by the back beam is 19,100 lb. per foot run (*Fig.* 11.9) and the beam spans 15 ft. from counterfort to counterfort.

Reversed $M = \frac{1}{12} \times 19{,}100 \times 15^2 \times 12 = 4{,}300{,}000$ lb.-in.
Shear $= 19{,}100 \times 7{\cdot}5 = 143{,}000$ lb.

If the beam is 24 in. wide at the bottom by 48 in. deep overall the reversed moment comes on an inverted L-section. With a resistance moment of $60bd^2$,

$$b = \frac{4{,}300{,}000}{60 \times 48^2} = 31 \text{ in. only.}$$

The lever arm will be about $0{\cdot}865 \times 44$ or 38 in.

The total shear stress is $\dfrac{143{,}000}{24 \times 38} = 157$ lb. per sq. in.

A_{st} for reversed $M = \dfrac{4{,}300{,}000}{12{,}000 \times 38} = 9{\cdot}45$ sq. in.

A_{st} for positive $M = \dfrac{2{,}150{,}000}{18{,}000 \times 38} = 3{\cdot}14$ sq. in.

In such a deep beam, which is deeper than one-quarter of the span, the writer would not hesitate to stress the shear steel to 18,000 lb. per sq. in. Using pairs

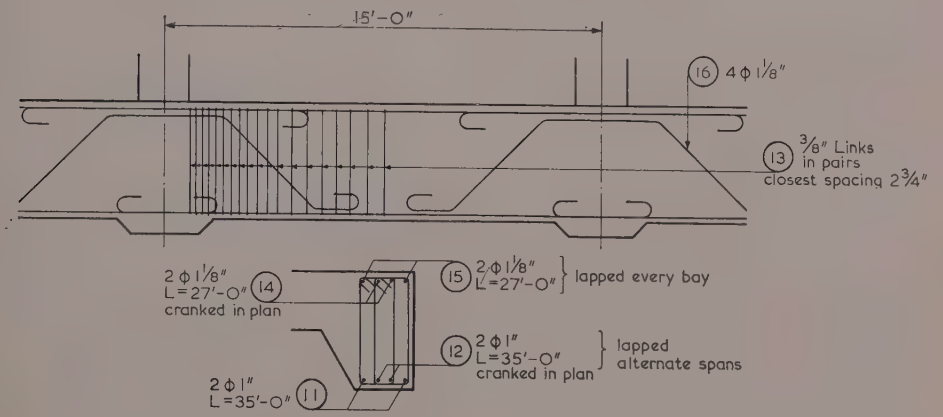

15'-0"
16 4 Φ 1⅛"
13 ⅜" Links in pairs closest spacing 2¾"
2 Φ 1⅛" L=27'-0" 14 cranked in plan
15 2 Φ 1⅛" L=27'-0" } lapped every bay
12 2 Φ 1" L=35'-0" cranked in plan } lapped alternate spans
2 Φ 1" L=35'-0" 11

Fig. 11.13.

of ⅜-in. diam. links with a minimum clear distance of 2 in. between pairs a spacing of 2¾ in. is obtained, giving a shear strength of $\dfrac{4 \times 0{\cdot}11 \times 38 \times 18{,}000}{2{\cdot}75} = 109{,}200$ lb.

This leaves a deficit of 143,000 — 109,200 = 33,800 lb. To cover this, instead of the foregoing arrangement, pairs of ½-in. diam. links at 3¾-in. centres or ⅜-in. diam. links in sets of three at 3⅛-in. centres could be used; (neither of these arrangements is very attractive), or additional bent-down bars can be provided to take the balance of 33,800 lb. With bars stressed to 12,000 lb. per sq. in. and inclined at 45 deg., an area of $\dfrac{33{,}800 \times \sqrt{2}}{12{,}000} = 3{\cdot}99$ sq. in. is needed; say four 1⅛-in. diam. bars.

A preliminary design for this beam is shown in *Fig.* 11.13. The arrangement would be improved by providing four 1⅛-in. diam. bars in the top and four 1-in. diam. bars in the bottom and welding them into long continuous lengths, adding 4 short bars at each counterfort.

When the reservoir is empty the earth pressure imposes an upward load of 7,920 lb. per foot on this beam (*Fig.* 11.11).

Reversed $M = \frac{1}{12} \times 7{,}920 \times 15^2 \times 12 = 1{,}780{,}000$ lb.-in.

$A_{st} = \dfrac{1{,}780{,}000}{18{,}000 \times 38} = 2{\cdot}61$ sq. in.

Counterforts

A preliminary design for the counterforts, which are spaced at 15-ft. centres, is shown in *Fig.* 11.14, with details in *Figs.* 11.16 and 11.17.

Taking a horizontal section 15 ft. below the water level, the stability of the part above this section is as shown in *Fig.* 11.15.

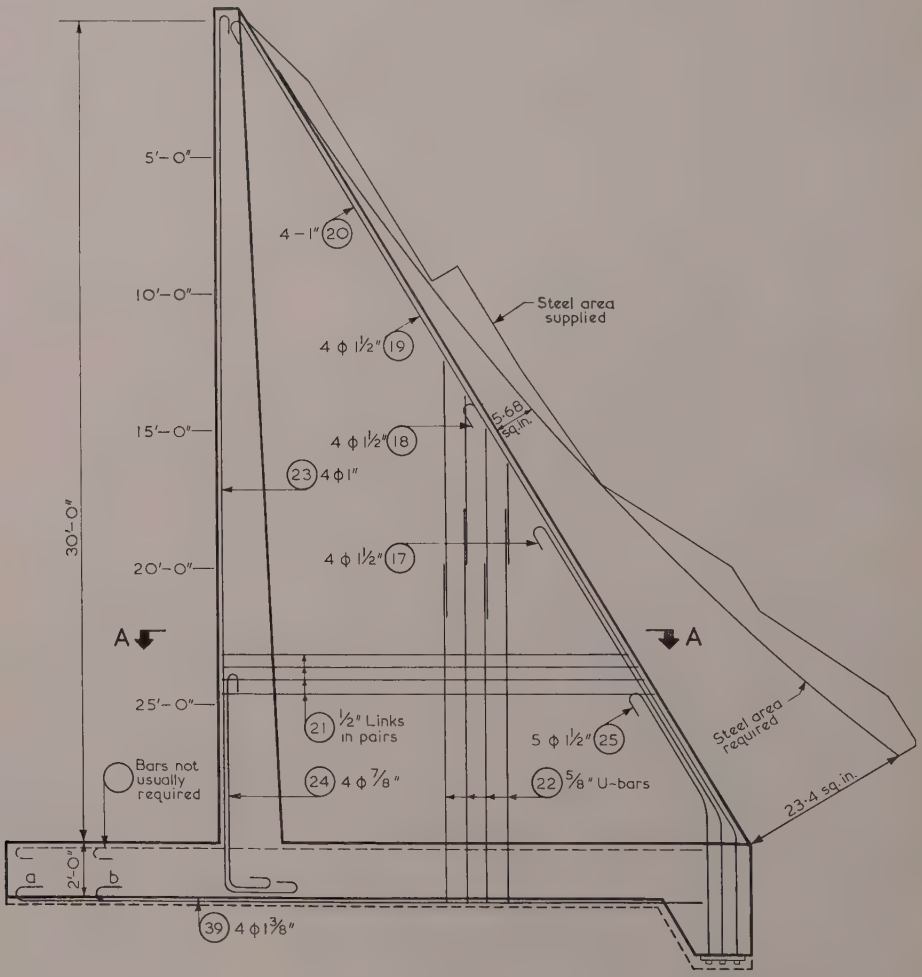


Fig. 11.14.

Water pressure $= \frac{1}{2} \times 62{\cdot}5 \times 15^2 \times 15 = 105{,}000$ lb. This is resisted by compression in the wall slab, shear on the horizontal section and tension in the back steel.

Taking moments about the centre of compression,

$$T \times 7{\cdot}75 \text{ ft.} = 105{,}500 \times 5{\cdot}0 \text{ ft.}$$
$$T = 68{,}000 \text{ lb.}$$

At a stress of 12,000 lb. per sq. in., 5·68 sq. in. of reinforcement is required.

Resolving horizontally, $F = 105{,}500 - T\cos\theta$

$= 105{,}500 - 34{,}900 = 70{,}600$ lb.

The compression in the wall slab C (by taking moments about the point where F and T intersect) is $\dfrac{105{,}500 \times 5{\cdot}0}{9{\cdot}0} = 58{,}500$ lb.

To check the tension in the concrete consider the section Z–Z in *Fig.* 11.15. From *Fig.* 11.2 the section modulus for tension is about $0{\cdot}25BD^2$, where D is 102 in.

$$M = 105{,}500 \times 60 = 633{,}000 \text{ lb.-in.}$$

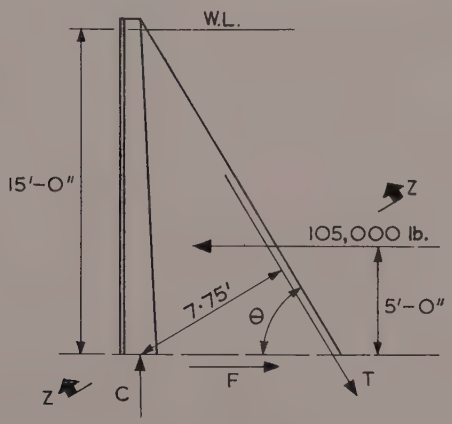


Fig. 11.15.

At a stress of 300 lb. per sq. in., minimum $B = \dfrac{633{,}000}{300 \times 0{\cdot}25 \times 102^2} = 8{\cdot}1$ in.

The counterfort will normally be the same thickness all the way up (see later). Repeating the process for the section at a depth of 30 ft;

Water pressure $= \frac{1}{2} \times 62{\cdot}5 \times 30^2 \times 15 = 422{,}000$ lb.

$T \times 15{\cdot}0$ ft. $= 422{,}000 \times 10{\cdot}0$ ft.

$T = 281{,}000$ lb.

The theoretical wall in *Fig.* 11·7 gives

$$T = 0{\cdot}332wH^2L = 0{\cdot}332 \times 62{\cdot}5 \times 30^2 \times 15 = 280{,}000 \text{ lb.}$$

At 12,000 lb. per sq. in., this needs 23·4 sq. in. of steel. The value of D on Section Y–Y is 195 in. Thus, minimum $B = \dfrac{422{,}000 \times 120}{300 \times 0{\cdot}25 \times 195^2} = 17{\cdot}75$ in.

This is smaller than the preliminary estimate. The difference is due to the fact that the theoretical wall slab has no thickness. In practice the thickness at the base of the wall is 25 in. which increases the theoretical value of D on Section Y–Y in *Fig.* 11.7 from $0{\cdot}498 \times 30{\cdot}0 \times 12 = 180$ in. to 195 in.

In the actual detail in *Fig.* 11.16 the counterfort is made 22 in. in thickness to give clearance between the main bars.

$$F \text{ at depth of } 30 \text{ ft.} = 422{,}000 - T\cos\theta$$
$$= 422{,}000 - (281{,}000 \times 0{\cdot}513) = 278{,}000 \text{ lb.}$$

The horizontal section at the base of the counterfort is 22in. in width and about 19 ft. deep. Treating it as a rectangular beam section with an effective depth of 18 ft. the lever-arm will be about $0{\cdot}865 \times 18 \times 12 = 187$ in.

$$\text{Shearing stress} = \frac{278{,}000}{22 \times 187} = 67{\cdot}5 \text{ lb. per sq. in.}$$

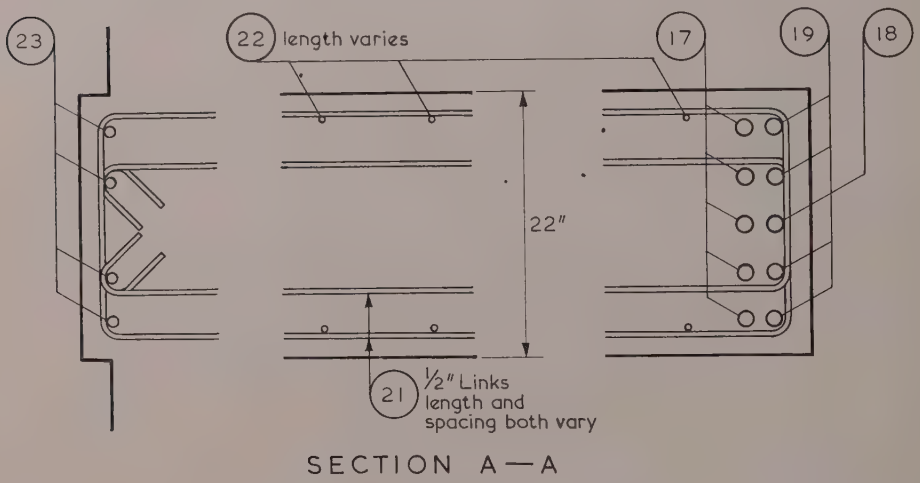


Fig. 11.16.

Repeating these calculations at depths of 5 ft., 10 ft., 20 ft. and 25 ft., the curve shown in *Fig.* 11.14, which indicates the area of main steel required at all depths, is obtained.

For a small wall the bottom ends of the main bars may be anchored by taking them in a long loop down to the bottom of the back beam and back again up into the counterfort, but for larger walls it is better to provide some kind of positive anchor plate, as shown in *Fig.* 11.17.

Sufficient horizontal steel must be supplied to prevent the wall slab tearing away from the counterfort. Neglecting secondary effects the maximum pressure per foot height of the wall is $30 \times 62{\cdot}5 \times 15 = 28{,}100$ lb. Part of this comes directly on the back of the counterfort and only $30 \times 62{\cdot}5 \times (15 - 1{\cdot}83)$ or 24,700 lb. is taken by the steel.

$$A_{st} = \frac{24{,}700}{12{,}000} = 2{\cdot}06 \text{ sq. in.}$$

Using $\frac{1}{2}$-in. diam. links in pairs (see *Fig.* 11.16), each pair has an area of 0·785 sq. in. At the bottom of the wall a spacing of $4\frac{1}{2}$ in. gives 2·1 sq. in. per foot of height. Halfway up the wall the spacing can be 9 in.

The minimum vertical steel is 0·3 per cent. or $\frac{0{\cdot}3}{100} \times 22 \times 12 = 0{\cdot}79$ sq. in. per foot. Using $\frac{5}{8}$-in. bars at 9-in. centres near both faces gives 0·82 sq. in. These could be U-bars in the lower part of the counterfort lapping with inverted U-bars in the upper part of the counterfort.

The long bars running up the back of the counterfort present a difficulty but they may be in shorter lengths connected by screw couplers. The sectional area

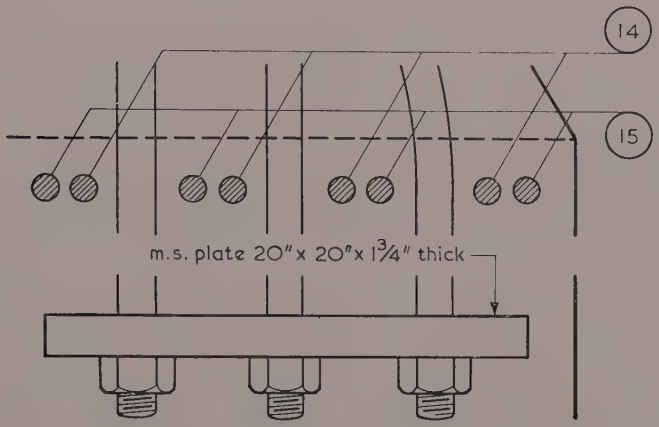


Fig. 11.17.

of a $1\frac{1}{2}$-in. diam. screwed rod at the bottom of the thread is 1·30 sq. in. so the tensile stress therein will not exeed 16,300 lb. per sq. in.

Vertical main bars are required up the front (earth) face of the counterfort to resist the moment produced by the earth pressure. Four long 1-in. diam. bars running the whole height and four short $\frac{7}{8}$-in. diam. bars terminating 6 ft. above the base slab working at 18,000 lb. per sq. in. will be sufficient to resist this moment. A preliminary design for this reinforcement is shown in *Fig.* 11.14. With *Figs.* 11.5, 11.12, 11.13, 11.16 and 11.17 this completes the preliminary design for the wall.

Secondary Effects

If there were no need to produce a watertight structure, the details in *Figs.* 11.12, 11.13 and 11.14 would make a satisfactory wall, but if the deflections and joint movements are considered it can be seen at once that the wall could not function as designed without the serious risk of cracking occurring.

A sectional plan of the wall in *Fig.* 11.8 at the base of the counterforts is shown in *Fig.* 11.18. In the preliminary calculations it was assumed that the base slab behind the wall slab spans freely 17 ft. from the wall slab to the back beam, but it cannot possibly do this as it is attached to the counterforts and some of the load must tend to span 15 ft. from counterfort to counterfort, particularly as the base slab is continuous under the counterforts and only freely supported by the back beam. This effect would be intensified if the spacing of the counterforts were reduced. Were it not for the strong twisting action caused by the cantilever toe the tail could be designed as a mesh panel slab spanning 15 ft. by 17 ft., but this large moment

applied to the edge of the panel must upset the usual moment coefficient of $\frac{1}{24}W$ or $\frac{1}{30}W$ (see *Figs.* 13.10, 13.11 and 13.12).

In *Fig.* 11.19, the joint between the wall slab and the base slab is shown. The strong cantilever action of the toe tends to rotate the joint as shown by the broken

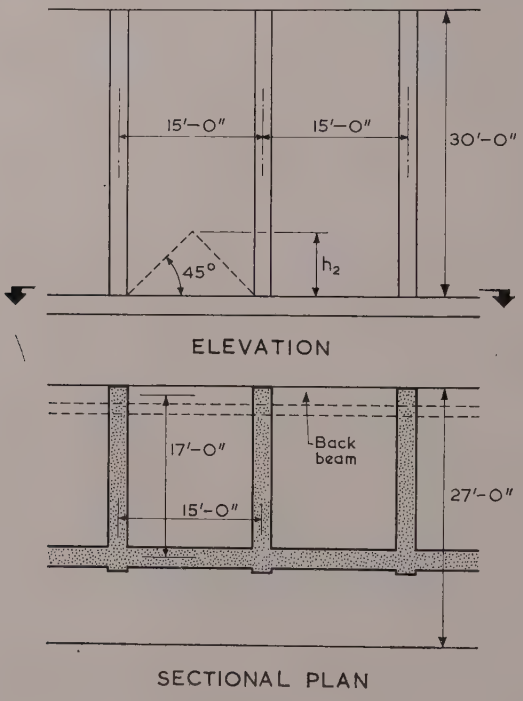


Fig. 11.18.

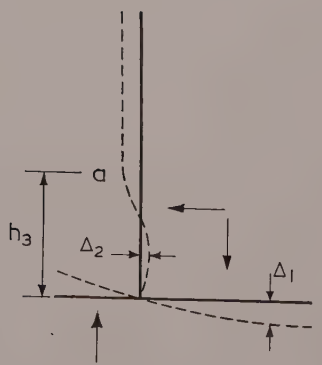


Fig. 11.19.

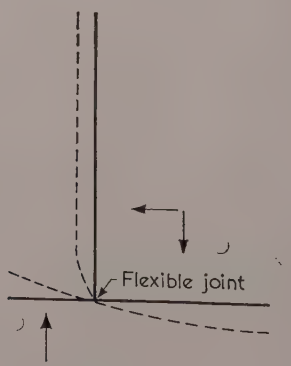


Fig. 11.20.

lines, swinging the wall slab back and the tail down, but both the wall slab and the tail are monolithic with the counterforts and both are loaded, so that they will deflect into a curve similar in type to the restraint curves in a circular tank. This deflection will produce reactions resisting the rotation. In the tail this reaction will depend on the thickness of the base slab, the span between counterforts and the

deflection Δ_1. In the wall slab the reaction will depend on the thickness of the wall slab, the span between counterforts and the deflection Δ_2.

If the wall slab is much thinner than the base slab or if it is severed from the base slab by a flexible joint (or open crack), it will impose no appreciable moment on the base slab. Assuming that such conditions do exist, the longitudinal bending of the tail is now considered as a separate problem (see *Fig.* 11.20).

The transcendental-exponential curve shown in *Fig.* 11.20 into which the tail actually deflects is too complicated for practical design, but some idea of the magnitude of the effect can be obtained by making a simple approximation and replacing this curve by a parabola.

If the back beam is omitted and the support that it provided is replaced by a

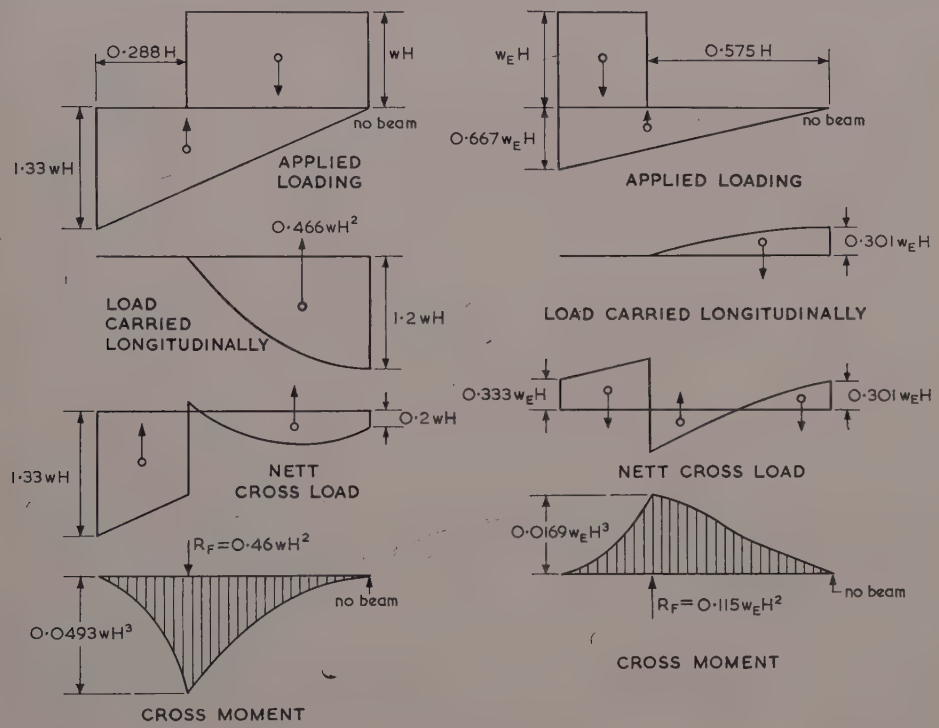


Fig. 11.21.

parabolic load supplied by the base slab acting as a beam, the loading shown in *Fig.* 11.7 is replaced by that in *Fig.* 11.21 and the moment acting from front to back in the base slab in *Fig.* 11.7 is modified to the moment shown in *Fig.* 11.21.

In *Fig.* 11.7 the back beam supplies a support of $0{\cdot}288wH^2$ at a distance of $0{\cdot}575H$ from R_F. The centre of pressure of the parabola in *Fig.* 11.21 is $0{\cdot}625 \times 0{\cdot}575H$ from R_F and the total pressure is

$$\frac{0{\cdot}288wH^2 \times 0{\cdot}575H}{0{\cdot}625 \times 0{\cdot}575H} = 0{\cdot}466wH^2.$$

The maximum intensity is $\dfrac{0{\cdot}466wH^2}{0{\cdot}575H} \times \dfrac{3}{2} = 1{\cdot}2wH$, as shown on the left-hand side of *Fig.* 11.21.

The rear end of the tail has therefore to carry a vertical load of 20 per cent. more than the horizontal pressure on the lowest part of the wall slab. If the effective height of the wall is 32 ft. and the counterforts are spaced at 15-ft. centres, the longitudinal moment in the tail of the base slab is

$1{\cdot}2wH \times L^2 \times \frac{1}{12} = 1{\cdot}2 \times 62{\cdot}5 \times 32 \times 15^2 \times \frac{1}{12} = 45{,}000$ lb.-ft. per foot width.

With a resistance moment of $60bd^2$, $d = \sqrt{\frac{45{,}000 \times 12}{60 \times 12}} = 27{\cdot}4$ in.

The cross moment due to water pressure is $0{\cdot}0493 \times 62{\cdot}5 \times 32^3 = 101{,}000$ lb.-ft. per foot width.

With a resistance moment of $250bd_1^2$, $d_1 = \sqrt{\frac{101{,}000 \times 12}{250 \times 12}} = 20{\cdot}1$ in.

The cross moment due to earth pressure (*Fig.* 11.21) is $0{\cdot}0169 \times 100 \times 32^3$ or 55,400 lb.-ft. for $k = 0{\cdot}249$.

For $k = 0{\cdot}2$, it will be about 44,500 lb.-ft. These figures apply to the thin, weightless, theoretical wall considered previously, but the figures for a practical wall of the same proportions would not be more than an inch or so different.

Taking the figures for the actual wall in *Fig.* 11.8, the value of R_B at a distance

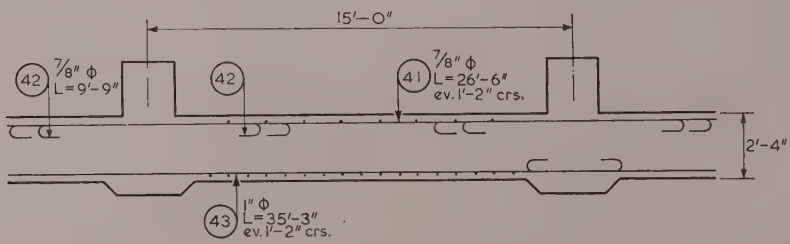


Fig. 11.22.

of 17 ft. from R_F is 19,100 lb. (see *Fig.* 11.9). The centre of pressure of the parabolic load that will replace it is $0{\cdot}625 \times 18$ or 11·25 ft. from R_F.

The total parabolic load is $\frac{19{,}100 \times 17}{11{\cdot}25} = 28{,}800$ lb., and the maximum intensity is $\frac{28{,}800}{18} \times \frac{3}{2} = 2{,}400$ lb. per sq. ft. Taking the effective height as 32 ft., then $1{\cdot}2wH$ in *Fig.* 11.21 $= 1{\cdot}2 \times 62{\cdot}5 \times 32 = 2{,}400$ lb. per sq. ft., which checks with the foregoing.

Under earth pressure, the downward value of R_B in *Fig.* 11.11 is 7,920 lb. The maximum intensity for the equivalent downward parabolic loading is

$$\frac{7{,}920}{19{,}100} \times 2{,}400 = 995 \text{ lb. per sq. ft.}$$

This is for a value of k of 0·2. The value for the theoretical wall in *Fig.* 11.21 where $k = 0{\cdot}249$, is $0{\cdot}301w_EH = 0{\cdot}301 \times 100 \times 32 = 963$ lb. per sq. ft., which checks approximately. A longitudinal section through the base slab, taken near the rear end of the tail and based on the moments just calculated, is shown in *Fig.* 11.22. Bars 41 lap at each counterfort and there are two bars 42 in between. Bars 43 lap at every other counterfort.

With a heavy cantilever toe, if the spacing of the counterforts is much greater than the width of the tail of the wall, then a back beam as shown in *Fig.* 11.7 should

be provided. If the spacing of the counterforts is much less than the width of the tail the longitudinal moment should be taken on the slab as in *Fig.* 11.21. Intermediate ratios of counterfort spacing to slab width may be treated by either method or, possibly, by a compromise between them. So far the conditions have been assumed to be as in *Fig.* 11.20. Reverting to *Fig.* 11.19, if the thickness of the bottom part of the wall slab is comparable with the thickness of the base slab and the two are monolithic, then the cantilever moment in the toe must be shared between them, approximately in proportion to the relative values of d^3. The problem is at present (1967) not soluble accurately since any appreciable deformation of the base slab will redistribute the pressure on the ground and invalidate the assumption of a straight-line variation (which in any case is only a rough approximation).

In the design in *Figs.* 11.12 and 11.14, the thickness of the base slab is 24 in.,

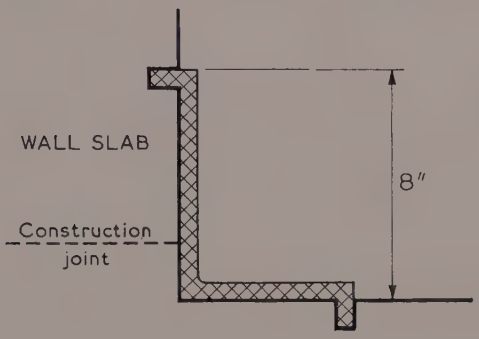


Fig. 11.23.

and of the wall 25 in. tapering upwards. If one-half of the cantilever moment is taken on each part,

$$M = \tfrac{1}{2} \times 116{,}500 \text{ lb.-ft.} = 698{,}000 \text{ lb.-in.},$$

and this, as far as the wall slab is concerned, produces tension on the liquid face. $\frac{M}{bd^2}$ for the wall slab $= \frac{698{,}000}{12 \times 25^2} = 93$, against an allowable figure of 60 (assuming that the limits specified in CP.2007:1960 are being observed). Even if the calculated tension in the concrete is ignored heavy vertical steel must be provided in the wall slab. If the thickness of the wall slab is increased no real advantage is gained as the proportion of the moment the wall has to carry increases as d^3. It is apparent that this design, although structurally quite sound is practically unworkable as soon as secondary effects and the possibility of serious cracking are considered.

There are a number of ways in which the problem of the secondary bending in the bottom part of the wall slab can be overcome.

1. Provide a joint similar to that shown in *Fig.* 2.8, running from counterfort to counterfort but not running through the counterforts. This expedient has already been mentioned in discussing *Figs.* 11.19 and 11.20. The writer would not adopt this solution.
2. Provide only nominal steel through the construction joint at the base of the wall slab, allowing this joint to open but cover it with an elastic flexible watertight membrane of asphalt, bitumen reinforced with glass-fibre scrim, synthetic resin, etc., as shown in *Fig.* 11.23. However, does any ‘ elastic flexible

watertight ' membrane exist that can be relied on to stay elastic, flexible and watertight for 40 or 50 years?

3. Increase the thickness of the base so that it takes a larger percentage of the moment. This is not very effective and increases the overall height of the wall.
4. Extend the counterfort outside the face of the wall, so that both the toe and the tail span from counterfort to counterfort. This is effective if the spacing of the counterforts is less than twice the width of the toe. With a wider

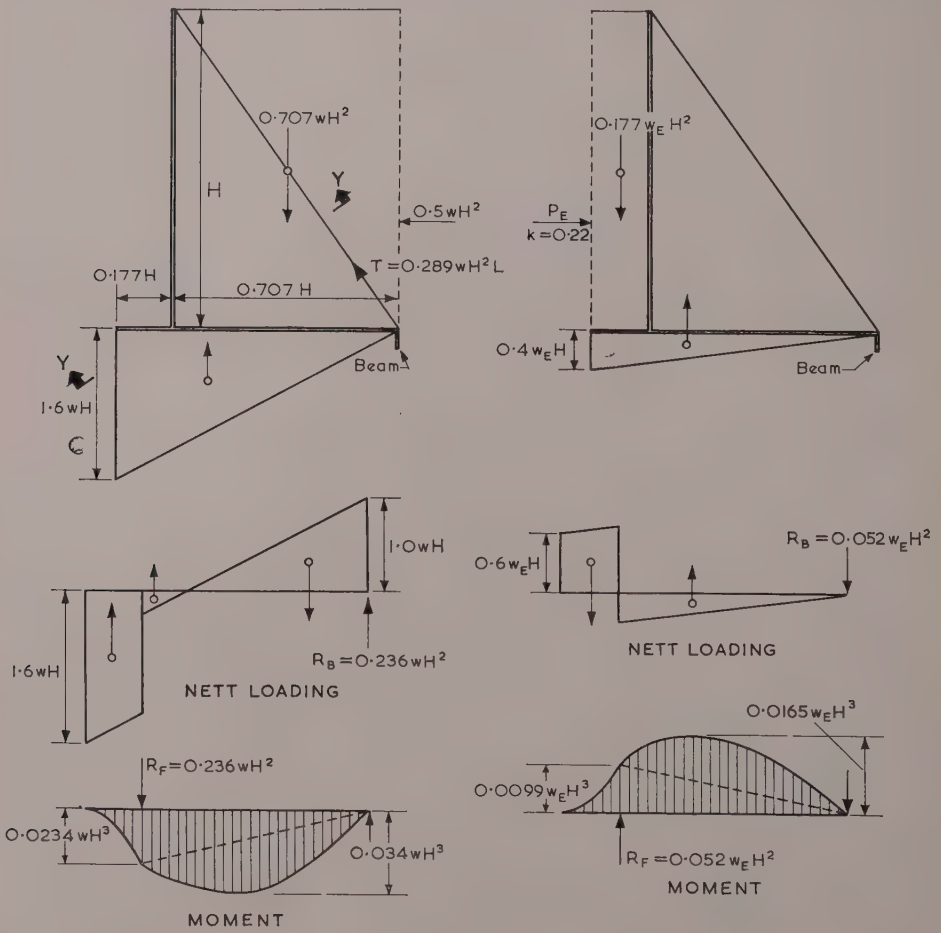


Fig. 11.24.

spacing and a shorter toe, cantilever action will develop half-way between counterforts.

5. Reduce the spacing of the counterforts to reduce the thickness of the wall slab. This is not really a separate issue as the designer must, in any case, prepare several complete preliminary designs with different counterfort spacings and select the best.
6. Vary the proportions of the wall by decreasing the width of the toe and increasing the width of the tail. This is usually the best remedy.

Consider the theoretical wall in *Fig.* 11.24. This is a thin, weightless wall

similar in type to the wall in *Fig.* 11.7, but having a toe equal to one-quarter of the width of the tail instead of one-half. The necessary width of base to bring the resultant inside the middle-third is $0{\cdot}884H$ (see *Fig.* 9.3). If a back beam is provided and the same primary assumption that all the loading on the base slab is carried by the slab spanning from front to back is made, the loadings and moments shown in *Fig.* 24 are obtained. Compared with *Fig.* 11.7 the maximum moment under water pressure has decreased from $0{\cdot}052wH^3$ to $0{\cdot}034wH^3$, and the maximum moment under earth pressure has decreased from $0{\cdot}0208w_EH^3$ to $0{\cdot}0165w_EH^3$. The maximum pressure on the ground, which is the sum of the pressures caused by water and earth loading has changed from $(1{\cdot}33wH + 0{\cdot}667w_EH)$ to $(1{\cdot}6wH + 0{\cdot}4w_EH)$. If $w = 62{\cdot}5$ and $w_E = 100$ lb. per sq. ft., this represents a decrease of 7 per cent.

If the back beam is omitted and the tail is arranged to span from counterfort to counterfort, the loadings in *Fig.* 11.25 are obtained, which compare directly with

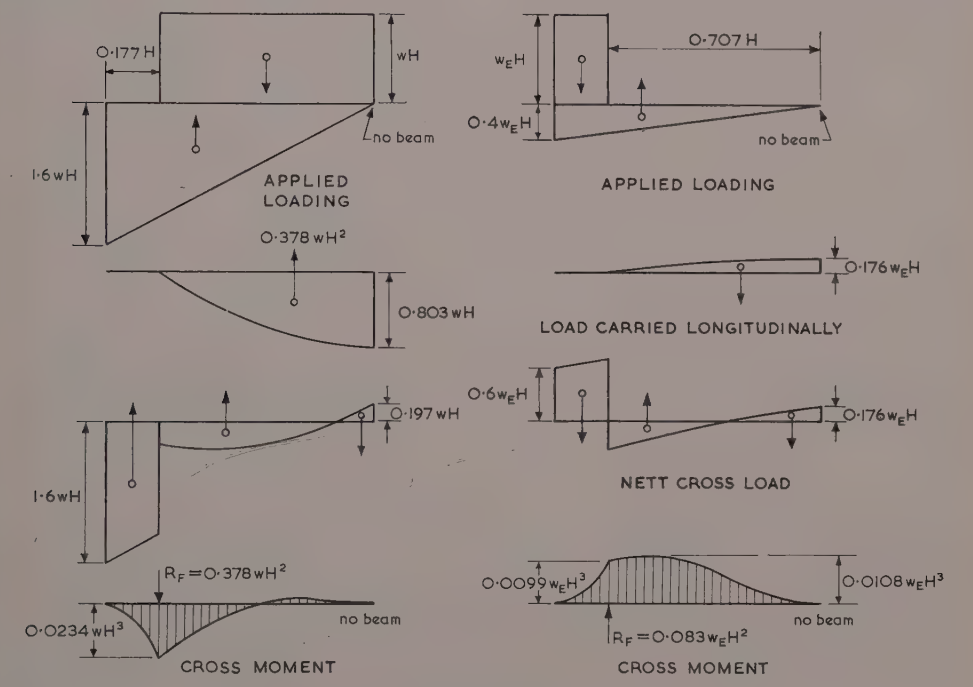


Fig. 11.25.

those shown in *Fig.* 11.21, and here again there is an all-round decrease in moments. Taking an effective height H of 32 ft., the maximum front-to-back moment causing tension on the underside of the slab is $0{\cdot}0234wH^3 = 0{\cdot}0234 \times 62{\cdot}5 \times 32^3 = 48{,}000$ lb.-ft. With a resistance moment of $250bd_1^2$, $d_1 = 13{\cdot}85$ in. or, say 16 in. overall. The maximum cross moment when the tank is empty is $0{\cdot}0108w_EH^3$. If w_E is 100 lb. per cu. ft.,

$$M = 0{\cdot}0108 \times 100 \times 32^3 = 35{,}400 \text{ lb.-ft.}$$

With a resistance moment of $60bd^2$, $d = 24{\cdot}3$ in. or, say 25 in. overall.

The maximum load carried longitudinally is $0{\cdot}803wH$. If the counterforts are at 15-ft. centres the moment will be $0{\cdot}803 \times 62{\cdot}5 \times 32 \times 15^2 \times \frac{1}{12} = 30{,}100$ lb.-ft. With a resistance moment of $60bd^2$, $d = 22{\cdot}4$ in. If, therefore, the width of the

toe is reduced to $0{\cdot}117H$, still keeping the counterforts at 15-ft. centres, a base slab of about 25 in. in thickness is required, which is the same thickness as the bottom of the wall slab. The cantilever moment on the toe is $0{\cdot}0234wH^3$. Dividing this equally between the base slab and the wall slab,

$$M = \tfrac{1}{2} \times 0{\cdot}0234 \times 62{\cdot}5 \times 32^3 = 24{,}000 \text{ lb.-ft. on each part.}$$

From *Fig.* 11.19, the wall slab spans horizontally above point *a*. Below this point it reverts to a cantilever. The applied loading at the bottom of this cantilever is $30 \times 62{\cdot}5 = 1{,}875$ lb. per sq. ft. Assuming the effective cantilever loading is parabolic as in *Fig.* 11.26,

$$M = \tfrac{2}{3} \times 1{,}875 \times h_3 \times 0{\cdot}375h_3 = 24{,}000 \text{ lb.-ft.}$$
$$h_3 = 7{\cdot}15 \text{ ft.}$$

Many early designers assumed an arbitrary value for h_3, such as 3 ft. Others, basing their opinion on the fact that the critical moments in mesh-panel construction occur across the diagonals, drew lines at 45 deg., as in *Fig.* 11.18, and assumed a cantilever

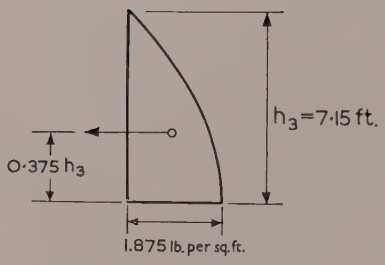


Fig. 11.26.

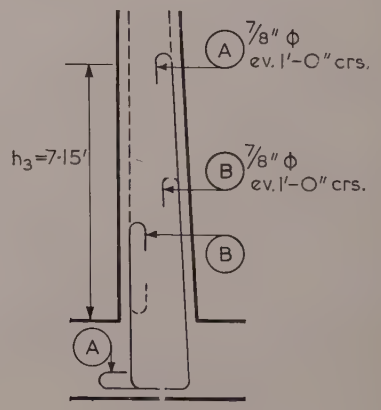


Fig. 11.27.

height of h_2 equal to one-half of the net spacing of the counterforts, which in this case is 6 ft. 7 in.

To resist a moment of 24,000 lb.-ft. with tension on the water face,

$$A_{st} = \frac{24{,}000 \times 12}{12{,}000 \times 0{\cdot}863 \times 22} = 1{\cdot}265 \text{ sq. in.; say } \tfrac{7}{8}\text{-in. diam. bars at 6-in. centres.}$$

Suspension steel to carry $R_F = 0{\cdot}083\ w_E H^2$ is needed when the tank is empty.

$$R_F = 0{\cdot}083 \times 100 \times 32^3 = 8{,}500 \text{ lb.}$$

$$A_{st} = \frac{8{,}500}{12{,}000} = 0{\cdot}71 \text{ sq. in.}$$

This tension only develops when the tank is empty and the steel required is not in addition to the area of 1·265 sq. in. which is only needed when the tank is full. A detail of the arrangement of this reinforcement is shown in *Fig.* 11.27. Eliminating the back beam necessitates modifying the detail at the bottom of the counterfort, as shown in *Fig.* 11.28. Considering the wall in *Fig.* 11.25, the loading of $0{\cdot}803wH$ is carried from counterfort to counterfort. If the net width between counterforts is (15 ft. less 1 ft. 10 in.), suspension steel to carry a maximum load of (13 ft. 2 in.)

$\times$ 0·803 $\times$ 62·5 $\times$ 32 is needed at each counterfort. The main steel in the back of the counterfort may be spread out as shown to pick up some of this load.
(13 ft. 2 in.) $\times$ 0·803 $\times$ 62·5 $\times$ 32 = 21,100 lb. per ft.

At a stress of 12,000 lb. per sq. in., a maximum area of steel of $\frac{21{,}000}{12{,}000} = 1{\cdot}76$ sq. in. is required. The detail in *Fig.* 11.28 shows $\frac{3}{4}$-in. diam. U-bars at 6-in. centres.

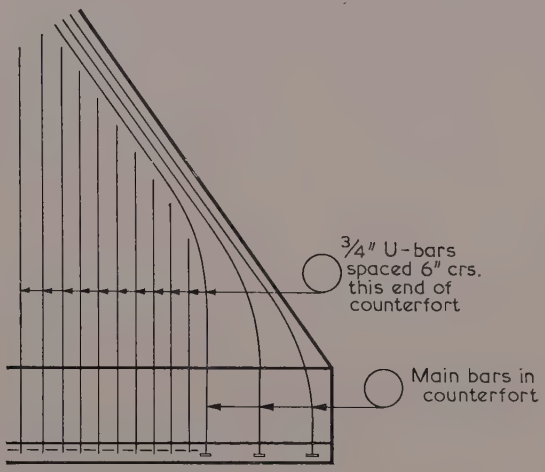


Fig. 11.28.

Some walls have been built without an external toe. Such walls rely for their stability against earth pressure on the friction developed between the earth banking and the external face of the walls. Since the amount of friction developed is variable and unpredictable such designs are best avoided.

Some Constructional Details

The arrangement of steel in the wall slab is straightforward. The layout of steel in the counterfort is complicated, with closely-spaced bars running horizontally and vertically and heavy rods running up the sloping edge. One of the problems is to provide proper anchorage and grip length without overcrowding the bars. The type of connection shown in *Fig.* 11.29 always requires careful detailing if cracks are to be avoided. The normal detail drawings to a scale of $\frac{3}{8}$ in. or $\frac{1}{2}$ in. to 1 ft. look reassuring, as they show the links fitting snugly round the main bars but this is misleading. It is standard practice to bend round a mandrel whose radius is twice the bar diameter. A $\frac{3}{8}$-in. diam. link will fit tight round a $1\frac{1}{2}$-in. diam. bar but a $\frac{1}{2}$-in. diam. link will not fit tightly round any bar used in everyday construction.

A tolerance of $+0$ to $-\frac{1}{4}$ in. is usual in bending $\frac{3}{8}$-in. diam. links and the main bar may well be $\frac{1}{8}$ in. clear of the link. In the particular case of a strongly-tapered counterfort, the links may be allowed to drop until they are tight against the main bars. In a very deep beam or wide counterfort they may be formed in two pieces, in which case also they can be placed tight against the main bars. The horizontal links in the bottom of a counterfort 30 ft. high may be more than 20 ft. long, a bar more than 40 ft. long being required if they are to be made in one piece, and they would normally be in two halves as shown in *Fig.* 11.30.

Using an arrangement where all the links bear tight against the main bars, the anchorage may be increased by welding the links to the main bars but this is a slow and expensive process. The bars must be held exactly in position during welding. A steel skeleton welded out of alignment is very difficult to get back into shape.

In *Fig.* 11.29, tension in the links on Section $x - x$ is resisted by:

1. Adhesion on the grip length l_g.
2. Resistance to crushing of the concrete inside the bend.
3. Anchorage (if any) provided by the main bars.

The resistance to crushing on a small area deeply embedded is very high, provided the concrete is sound. For a horizontal link or the bottom end of a vertical link there should be no question of its strength, but at the upper end of a vertical

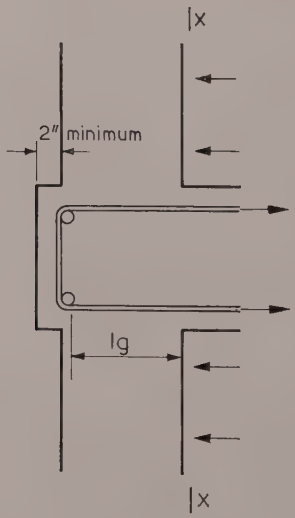


Fig. 11.29.

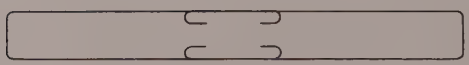


Fig. 11.30.

link close contact between the concrete and the underside of the bar may be spoiled by the use of harsh, over-dry or over-wet concrete. If necessary the value of l_g may be increased by increasing the projection, shown as 2 in. minimum in *Fig.* 11.29.

In the detail in *Fig.* 11.16, $\frac{1}{2}$-in. diam. links are used to hold the wall slab to the counterfort. Assuming that a grip length of 30 diameters is required for bars stressed to 12,000 lb. per sq. in. and ignoring the assistance provided by items (2) and (3), $\frac{1}{2}$-in. diam. links can certainly be used where the thickness of the wall is 15 in. and $\frac{3}{8}$-in. diam. links where the wall is 11 in. thick, and it would be possible to use $\frac{5}{8}$-in. diam. links where the wall is 19 in. in thickness. Using $\frac{5}{8}$-in. diam. links at the bottom of the wall would increase the spacing from $4\frac{1}{2}$ in. to 7 in.; this is no great advantage. Spacings of less than 4 in. should be avoided if possible.

If the thickness of the base slab is 24 in., the use of $\frac{3}{4}$-in. diam. U-bars stressed to 12,000 lb. per sq. in. is quite safe to hold the base slab up to the counterfort (see *Fig.* 11.28).

If the bars are too large to anchor by adhesion or by end hooks, they may always be held by anchor plates as shown in *Fig.* 11.17.

The main bars in *Fig.* 11.16 are arranged to have a clear space of 3 in. between them. This could be reduced to 2 in. as a minimum.

Counterforts on Rock Foundations

On a rock foundation the counterforts may be supported by a raking strut (or struts) as shown in *Fig.* 11.31. The tendency of the wall to lift bodily may be

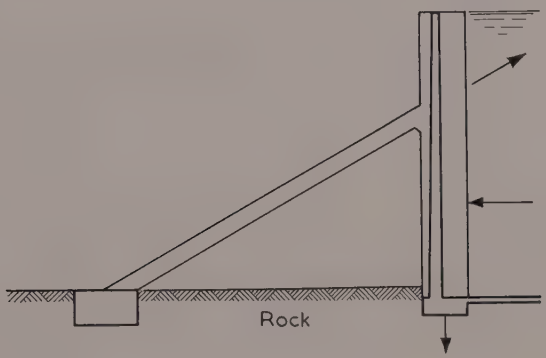


Fig. 11.31.

resisted by increasing its weight, by tying it down into the rock, or by providing a base slab to pick up sufficient vertical loading. Under earth pressure the strut becomes a tie and must be anchored down. Alternatively a second raking strut inside the tank could be provided.

Sloping Walls

The high cost of counterforts in tension (particularly if they are designed to CP.2007:1960) naturally leads to the consideration of providing sloping walls

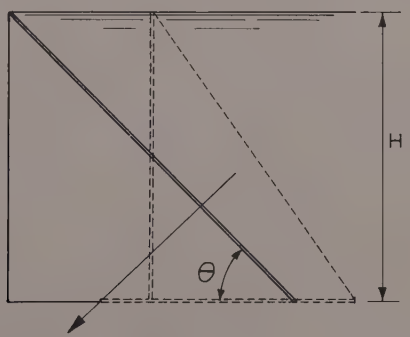


Fig. 11.32.

with the counterforts in compression, as shown in *Fig.* 11.32. With the theoretical weightless wall as shown in *Fig.* 11.32, the centre of pressure on the ground passes through the middle-third point of the base when $\theta = 45$ deg. Comparing this with the theoretical wall in *Fig.* 11.24, which is shown by broken lines in *Fig.* 11.32,

a nearly 42-per-cent. greater wall slab will be required but some of the thick base slab will be saved. The counterforts may be larger in area but may be thinner and more lightly reinforced. The main difficulty lies in the shuttering for the sloping wall, but the cost of this may be kept within reasonable limits by a properly designed shuttering fit-up. The scheme is most attractive on a rock foundation where the counterfort requires only a nominal width of footing.

Other Shapes of Counterfort

So far only the elementary triangular shape has been discussed. In this shape both the horizontal and vertical bars vary in length and the compacting of the concrete round the main bars is made somewhat difficult by the sloping shuttering to the inside edge of the counterfort. Both these minor difficulties may be eliminated

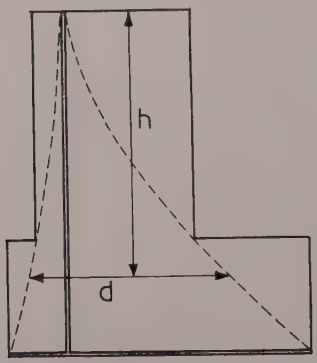


Fig. 11.33.

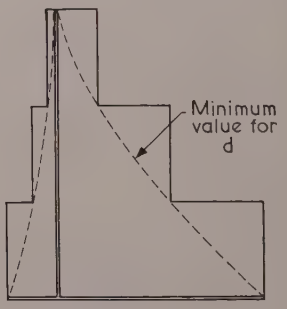


Fig. 11.34.

by using a counterfort of the type shown in *Figs.* 11.33 and 11.34. If the counterfort is of uniform thickness b and varying horizontal depth d, if it acts as a rectangular beam subjected to a moment of $0{\cdot}167wh^3L$ at any depth h, and if the moment of resistance is $60bd^2$, then

$$60bd^2 = 0{\cdot}167wh^3L, \text{ or } d \text{ varies as } \sqrt{h^3}.$$

This gives a concave curve (shown by broken lines in *Figs.* 11.33 and 11.34), and any series of vertical or sloping lines that approximate to this curve may be used. The elementary triangular shape is a first approximation. The shapes in *Figs.* 11.33 and 34 are second and third approximations. The beginner is advised to keep to the simpler shapes or he might find himself in a muddle with his steel arrangement.

The reader should study the counterfort shown in *Fig.* 11.14.

CHAPTER TWELVE

LARGE RESERVOIRS WITH WALLS SPANNING VERTICALLY

When a reservoir or tank is provided with a roof the first natural reaction of any designer is to connect the top of the wall to the edge of the roof and span from floor to roof. The roof structure is generally not sufficiently stiff to provide a fixed joint at the top of the wall, but the bottom end of the wall, when this consists of a simple slab, can generally be held in position and direction by the floor slab.

A cantilever of height H carrying a triangular load varying from 0 to wH has a maximum moment of $0{\cdot}167wH^3$. A freely-supported beam of height H and carrying the same load has a maximum moment of $0{\cdot}064wH^3$. A beam of height H and of uniform section, freely-supported at the top and fixed at the bottom and carrying this same load, has a moment of $0{\cdot}0667\ wH^3$ at the base, a positive moment of $0{\cdot}0297wH^3$, and a top reaction of $0{\cdot}1wH^2$. Thus, by connecting the top of the wall to the roof the maximum moment can be reduced to 40 per cent. of the cantilever moment, as far as primary stresses are concerned. Unfortunately the secondary effects (that is, the additional stresses due to the deflections and extensions of the structure) in a large tank or reservoir can cause such high stresses that the wall and roof should not be connected. Provision of expansion joints in the wall may necessitate their being carried across the roof, which would make it impracticable to carry tension across from wall to wall. To exploit the idea of spanning the wall vertically, the roof must be free of expansion joints.

Liquid Loading

Usually the water level is some distance below the underside of the roof and the loading is as shown in *Fig.* 12.1. In order to use standard moment coefficients

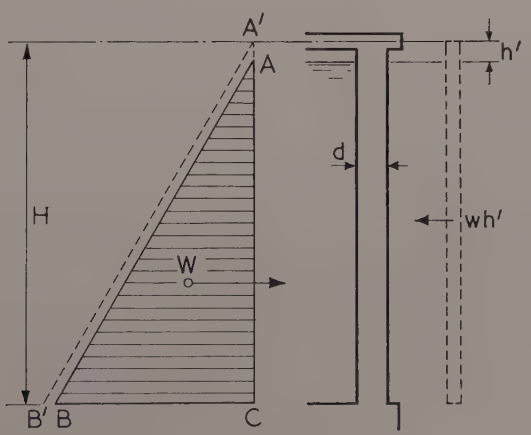


Fig. 12.1.

for tapering walls it is possible, for all practical purposes, to replace the triangular load ABC with the triangular load A′B′C and the negative uniform load equal to wh'.

Earth Loading

The wall of a large roofed reservoir surrounded by a substantial bank of earth is shown in *Fig.* 12.2. In mid-winter the thermal contraction of the roof will drag the top of the wall inwards, leaving a space between the back of the wall and the face of the bank. If the bank consists of dry granular material, it will follow the

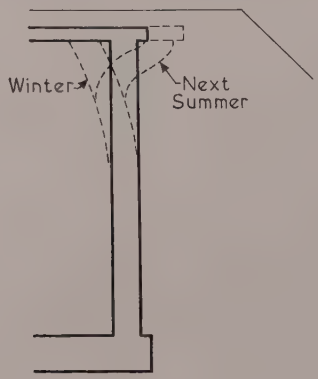


Fig. 12.2.

movement of the wall and fill this space. Next mid-summer the roof will expand but the wall will be held by the counterpressure from the ground which may be from 10 to 20 times the active pressure, and the wall may develop serious cracks or (more likely) the joint at the top of the wall may shear. This action may be prevented by inserting a substantial layer of compressible material (say 3 to 4 inches of foamed plastic) between the back of the wall and the earth.

Slab of Uniform Thickness Spanning Vertically

In *Figs.* 12.1 and 12.3 a wall slab of uniform thickness spanning a height H, fixed by the floor slab at the bottom and connected to the roof at the top is shown. The structural reaction at the top of the wall slab puts the roof in tension and causes it to stretch. This effect is generally much smaller than the thermal expansion and contraction of the roof.

The top joint of the wall will suffer a rotation θ and a horizontal displacement x due to stretching and thermal expansion of the roof. The bottom joint is assumed to be fixed in position and direction by the floor slab.

The applied loading is shown in *Fig.* 12.1.

$$M_{1F} = -\frac{WH}{15} + \frac{wh'H^2}{12} = A.$$

$$M_{2F} = +\frac{WH}{10} - \frac{wh'H^2}{12} = B.$$

Tabulating the effects:

	M_1	M_2
Joints fixed	A	B
Due to θ	$4EK\theta$	$2EK\theta$
,, ,, x	$6EK\frac{x}{H}$	$6EK\frac{x}{H}$

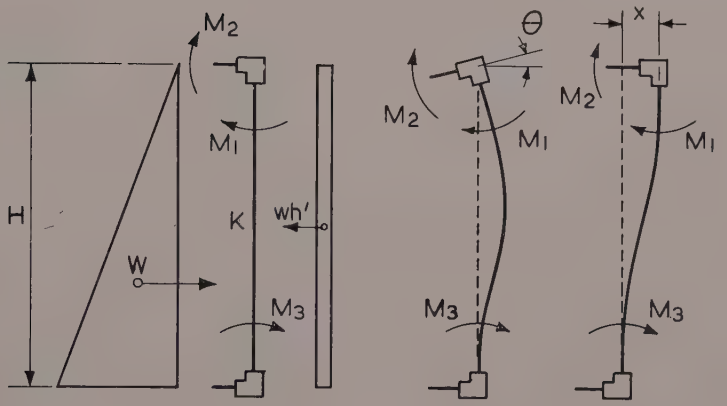


Fig. 12.3.

The roof construction is generally insufficiently stiff to fix the top joint in direction and, as a first approximation,

$$M_1 = M_2 = 0.$$

$$A + 4EK\theta + 6EK\frac{x}{H} = 0.$$

$$4EK\theta = -A - 6EK\frac{x}{H}.$$

$$M_3 = B + 2EK\theta + 6EK\frac{x}{H}$$

$$= B - 0{\cdot}5A - 3EK\frac{x}{H} + 6EK\frac{x}{H}$$

$$= B - 0{\cdot}5A + 3EK\frac{x}{H}.$$

Taking the term $3EK\frac{x}{H}$, which shows the moment at the base of the wall caused by the outward movement of the top joint, and writing $K = I/H$, $I = \frac{1}{12}bd^3$ and $Z = \frac{1}{6}bd^2$, the stress in the concrete (ignoring the reinforcement) due to this effect alone is $3E\frac{bd^3}{12H} \times \frac{x}{H} \times \frac{6}{bd^2} = 1{\cdot}5\frac{Exd}{H^2}$. If the distance x is taken as equivalent to a rise in temperature of 30 deg. F. in a roof 100 ft. in width and E is assumed to be 3,000,000 lb. per sq. in., $x = 30 \times 0{\cdot}000006 \times (50 \times 12) = 0{\cdot}108$ in.

If $H = 15$ ft. $= 180$ in., and $d = 16$ in., bending stress at base of wall due to x alone

$$= \frac{1{\cdot}5 \times 3{,}000{,}000 \times 0{\cdot}108 \times 16}{180^2} = 240 \text{ lb. per sq. in.}$$

The remainder of M_3 is $(B - 0{\cdot}5A)$

$$= + \frac{WH}{10} - \frac{wh'H^2}{12} + \frac{WH}{30} - \frac{wh'H^2}{24}$$

$$= + \frac{WH}{7{\cdot}5} - \frac{wh'H^2}{8}.$$

For a slab of uniform thickness all these are standard values, but the method may also be employed to solve the case of a tapering slab, which is dealt with later in this chapter.

$$W = \tfrac{1}{2} \times 62{\cdot}5 \times 15^2 = 7{,}030 \text{ lb.}$$

If $h' = 10$ in. then $wh' = 62{\cdot}5 \times 0{\cdot}83 = 52$ lb. per foot.

$$\text{Structural moment} = \frac{7{,}030 \times 15}{7{\cdot}5} - \frac{52 \times 15^2}{8} = 12{,}600 \text{ lb.-ft.}$$

Stress due to this moment (ignoring the reinforcement)

$$= 12{,}600 \times 12 \times \frac{6}{12 \times 16^2} = 295 \text{ lb. per sq. in.}$$

If this is added to the stress due to x, a total stress of $295 + 240 = 535$ lb. per sq. in. is obtained. Allowing for the reinforcement might reduce this to about 450 lb. per sq. in. or about 50 per cent. greater than the limiting value specified in CP.2007: 1960.

The total moment is $12{,}600 + 10{,}250 = 22{,}850$ lb.-ft.

If $d_1 = 14$ in., $\qquad \frac{M}{bd_1^2} = \frac{22{,}850 \times 12}{12 \times 14^2} = 116{\cdot}5.$

If the reinforcement were efficiently detailed and the tank built by experienced workmen the writer would accept a wall 16 in. in thickness, particularly if the reservoir is to be banked round with earth and the roof covered with 1 ft. of soil.

For any given values of x and H, the stress due to x alone varies directly as d. If d is increased to 18 in. the stress will be $(233 + 270) = 503$ lb. per sq. in. against $(295 + 240) = 535$ lb. per sq. in.

Tapering Slab Spanning Vertically

If a slab having a thickness varying from $0{\cdot}667d_c$ at the top to $1{\cdot}33d_c$ at the base is used, d_c being the thickness halfway down the slab as in *Fig.* 12.4, and the same loading as in *Fig.* 12.3 is applied,

$$M_{1F} = -0{\cdot}04WH + 0{\cdot}055wh'H^2 = G$$

and $$M_{3F} = +0{\cdot}136WH - 0{\cdot}12wh'H^2 = D.$$

From tables, $C_{1A} = 5{\cdot}75$, $C_{1B} = 2{\cdot}0$, and $C_{2A} = C_{2B} = 1{\cdot}7$.
Tabulating:

	M_1	M_2
Joints fixed	G	D
Due to θ	$2EK\theta$	$1{\cdot}7EK\theta$
„ „ x	$3{\cdot}7EK\frac{x}{H}$	$7{\cdot}45EK\frac{x}{H}$

Assuming $M_1 = 0$,

$$G + 2EK\theta + 3{\cdot}7EK\frac{x}{H} = 0.$$

$$2EK\theta = -G - 3{\cdot}7EK\frac{x}{H}.$$

$$M_3 = D + 1{\cdot}7EK\theta + 7{\cdot}45EK\frac{x}{H}$$

$$= D - 0{\cdot}85G - 3{\cdot}14EK\frac{x}{H} + 7{\cdot}45EK\frac{x}{H}$$

$$= D - 0{\cdot}85G + 4{\cdot}31EK\frac{x}{H}.$$

The last term is the moment at the base of the wall caused by the outward movement

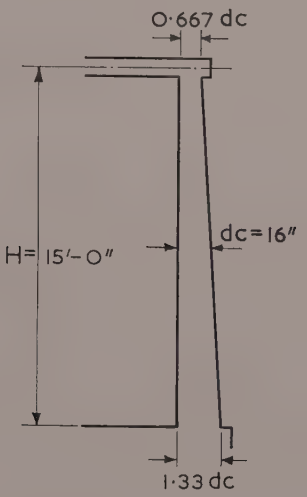


Fig. 12.4.

of the top joint. $K = I_c/H$, where $I_c = \frac{1}{12}bd_c^3$. Z at base of wall $= \frac{1}{6}b(1{\cdot}33d_c)^2$. The stress in the concrete (ignoring the reinforcement) due to this effect alone is

$$4{\cdot}31\frac{Ebd_c^3}{12H} \times \frac{x}{H} \times \frac{6}{b(1{\cdot}33d_c)^2} = 1{\cdot}21\frac{Ex}{H^2}d_c.$$

Again assuming $x = 0{\cdot}108$ in., $H = 180$ in. and $d_c = 16$ in., the wall is 21·3 in. thick at the base and 10·7 in. thick at the top.

Bending stress at base of wall due to x alone

$$= \frac{1{\cdot}21 \times 3{,}000{,}000 \times 0{\cdot}108 \times 6}{180^2} = 194 \text{ lb. per sq. in.}$$

The remainder of M_3 is $D - 0{\cdot}85G$

$$= +0{\cdot}136WH - 0{\cdot}12wh'H^2 + 0{\cdot}034WH - 0{\cdot}047wh'H^2$$
$$= +0{\cdot}17WH - 0{\cdot}167wh'H^2.$$

Again taking W as 7,030 lb. and wh' as 52 lb. per foot,

Structural moment $= +(0{\cdot}17 \times 7{,}030 \times 15) - (0{\cdot}167 \times 52 \times 15^2) = 15{,}950$ lb.-ft.

Stress due to this moment (ignoring the reinforcement)

$$= 15{,}950 \times 12 \times \frac{6}{12 \times 21{\cdot}3^2} = 210 \text{ lb. per sq. in.}$$

Adding this to the stress due to x, the total stress is $210 + 194 = 404$ lb. per sq. in. This is 75 per cent. of the calculated stress in a wall of uniform thickness having the same average thickness.

The total moment at the base is $15{,}950 + 14{,}750 = 30{,}700$ lb.-ft.

$$\frac{M}{bd^2} = \frac{30{,}700 \times 12}{12 \times 21{\cdot}3^2} = 67{\cdot}5,$$

which is still in excess of the maximum permitted by CP.2007:1960.

If $d_1 = 19{\cdot}3$ in., $\dfrac{M}{bd_1{}^2} = \dfrac{30{,}700 \times 12}{12 \times 19{\cdot}3^2} = 82{\cdot}4$. This is less than the value of $95\ bd_1{}^2$ which was employed as standard practice for designing work of this type in the early part of this century.

If the design is carried a stage further, still maintaining an average wall thickness of 16 in. but reducing the top to 8 in. and increasing the bottom to 24 in., the mid-summer moment at the base is 17,000 (structural) + 16,400 (temperature) = 33,400 lb.-ft. = 400,000 lb.-in.

$\dfrac{M}{bd^2} = \dfrac{400{,}000}{12 \times 24^2} = 58$, which is within the limits of CP.2007:1960.

The problem may be approached from the other direction. If a free-standing cantilevered wall 15 ft. high is subjected to a triangular load equal to 14 ft. 2 in. depth of water, the total lateral pressure is 6,280 lb. If the wall is 16 in. in thickness halfway up, the deflection at the top for a wall of uniform thickness (*Fig.* 6.4)

$$0{\cdot}0667\frac{WH^3}{EI_c} = \frac{0{\cdot}0667 \times 6{,}280 \times 180^3 \times 12}{3{,}000{,}000 \times 12 \times 16^3} = 0{\cdot}199 \text{ in.}$$

With a slab of the same average thickness but tapering from 2 to 1 the deflection is 0·113 in., and when tapering 3 to 1 the deflection is 0·098 in.

If the deflection at the top of the wall, when acting as a cantilever, is more than the thermal expansion of the roof then the roof will still afford partial support and this will reduce the moment at the base of the wall.

If the deflection at the top of the wall, when acting as a free cantilever, is less than the thermal expansion of the roof then the roof will push the top of the wall out further and increase the moment at the base of the wall.

In the four cases considered the moments at the base of the wall are as follows.

Slab of uniform thickness: 22,850 lb.-ft.
Slab tapering 2–1: 30,700 lb.-ft.
Slab tapering 3–1: 33,400 lb.-ft.
Free cantilevered slab: 29,600 lb.-ft.

Generally speaking there is little or no structural advantage in connecting the roof of a large reservoir to the top of the wall, but there may be other reasons for so doing and the design of the 3 : 1 tapered wall in *Fig.* 12.5 may be completed by adding the moments due to earth pressure when the reservoir is empty. The most severe effects from earth pressure will occur in mid-winter, the top of the wall being dragged inward by thermal contraction of the roof.

The loading is shown in *Fig.* 12.5 and consists of a triangular load of $0{\cdot}5\ pH^2$

and an added uniform load of ph'' per foot of height, where p is the earth pressure per sq. ft. per foot of depth.

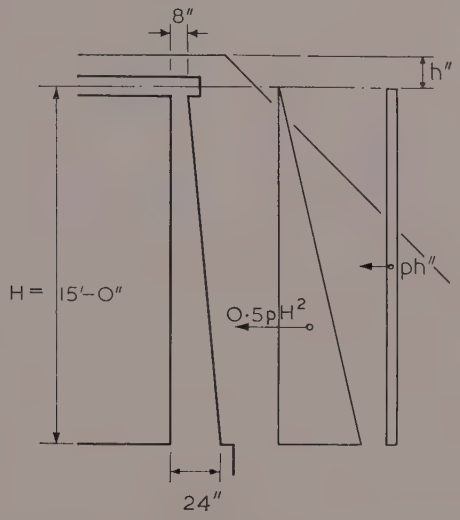


Fig. 12.5.

If $p = 20$ lb., with $H = 15$ ft. and $h'' = 1$ ft. 4 in.,

$$0{\cdot}5pH^2 = 0{\cdot}5 \times 20 \times 15^2 = 2{,}250 \text{ lb.}$$

$$ph'' = 20 \times 1{\cdot}33 = 26{\cdot}7 \text{ lb. per foot of height.}$$

$$M_{1F} = +(0{\cdot}026 \times 2{,}250 \times 15) + (0{\cdot}04 \times 26{\cdot}7 \times 15^2).$$

$$M_{3F} = -(0{\cdot}156 \times 2{,}250 \times 15) - (0{\cdot}145 \times 26{\cdot}7 \times 15^2).$$

Writing $x = -0{\cdot}108$ in. and solving,

$M_3 = -\ 7{,}280$ lb.-ft. (structural)

$\quad -16{,}400$ lb.-ft. (temperature)

$\quad -23{,}680$ lb.-ft.

If the wall were not attached to the roof and were acting as a simple cantilever the moment at the base would be

$$2{,}250 \times 5 \text{ ft.} + (26{\cdot}7 \times 15) \times 7{\cdot}5 \text{ ft.} = 14{,}250 \text{ lb.-ft.}$$

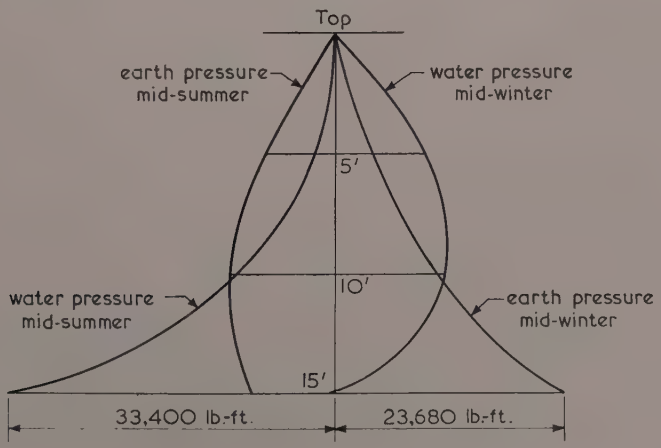


Fig. 12.6.

By connecting the top of the wall to the roof the mid-winter moment at the base of the wall has been increased by some 66 per cent.

If the loading and the moment at the base are known the top reaction R_T can at once be calculated by taking moments about the bottom of the wall.

R_T (pulling the top inward) $\times$ 15 ft. = 23,680 — 14,250 lb.-ft.
R_T = 628 lb. (mid-winter, tank empty).

The derivation of the bending-moment diagram follows at once. In *Fig.* 12.6 the moments in the 3 : 1 tapered wall in *Fig.* 12.5 are shown under the four major combinations of structural loading and temperature effects.

A preliminary design for the reinforcement in the wall is shown in *Fig.* 12.7, the steel near the water face being stressed to 12,000 lb. per sq. in. and that near the earth face to 18,000 lb. per sq. in.

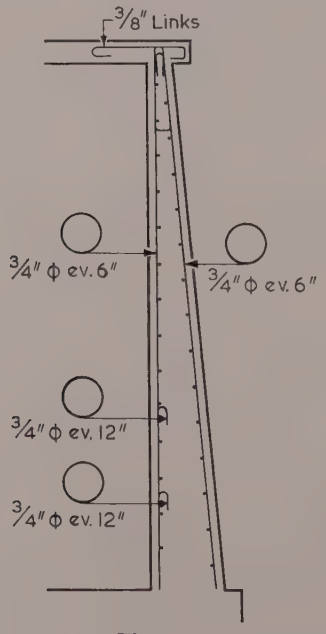


Fig. 12.7.

At the base of the wall:

$$A_{st} \text{ (inside)} = \frac{33{,}400 \times 12}{12{,}000 \times 0{\cdot}86 \times 22} = 1{\cdot}76 \text{ sq. in.;}$$

say $\frac{3}{4}$-in. diam. bars at 3-in. centres.

$$A_{st} \text{ (outside)} = \frac{23{,}680 \times 12}{18{,}000 \times 0{\cdot}86 \times 22} = 0{\cdot}832 \text{ sq. in.;}$$

say $\frac{3}{4}$-in. diam. bars at 6-in. centres.

When calculating the pressure on the ground the designer must not forget the reactions, vertical and horizontal, imposed on the top of the wall by the roof.

Owing to the strong outward thrust of the roof in summer it is difficult to make the centre of pressure fall within the middle-third of the base slab under all eight sets of conditions that must be checked. These are as follows: unbanked tank full

or empty, winter or summer, and banked tank full or empty, winter or summer. In *Fig.* 12.8 the necessary stability has been obtained by thickening the base slab and by picking up the first line of roof columns on the inward end.

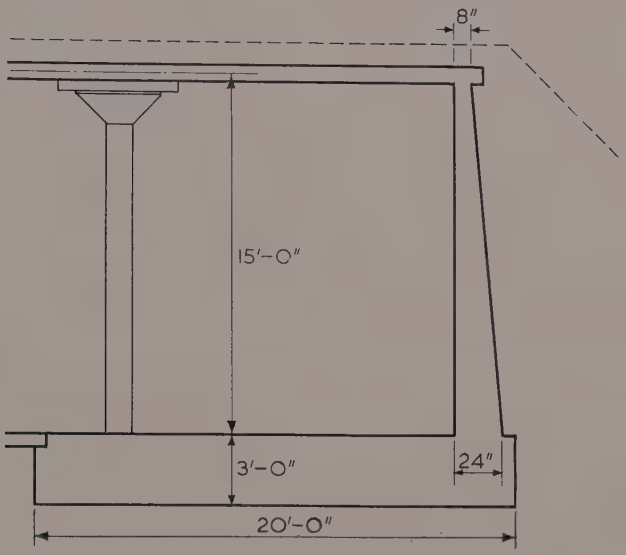


Fig. 12.8.

Longitudinal Movement

The roof will expand and contract in all directions. Inward and outward movement of the top of the wall will take care of the expansion and contraction in one direction. Thermal movement in the other direction may be accommodated by the provision of vertical expansion joints in the wall slab.

Slab and Beam Walls

Reservoir walls have been built as shown in *Fig.* 12.9 with vertical beams simply tied into the floor and roof. The primary stresses are simple but, as has already

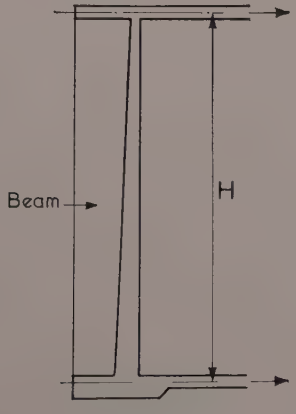


Fig. 12.9.

been seen, the top of the beam may rock backwards and forwards to such an extent that the joint between the bottom of the beam and the floor slab may crack and steadily deteriorate with each yearly cycle of temperature change. In the example already worked out, this movement is calculated as $\pm 0{\cdot}108$ in. On a rock foundation, where the bearing area can be reduced to a narrow strip, the detail in *Fig.* 12.10

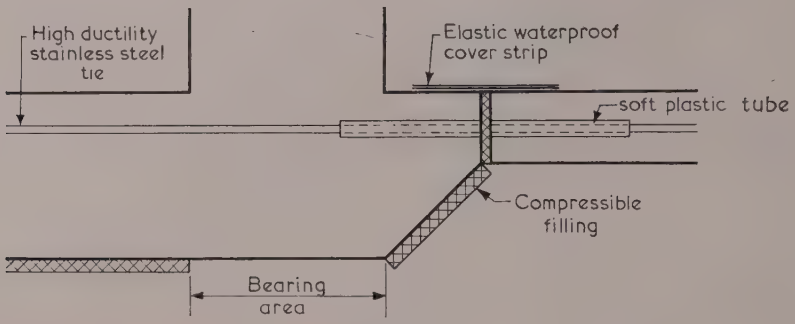


Fig. 12.10.

could be used. (The author personally dislikes details of this type but some experienced engineers use them.)

The danger is probably more apparent than real. Nevertheless the designer should give the problem close consideration and every reasonable precaution should be taken to reduce the range of temperature in the roof.

CHAPTER THIRTEEN

NARROW TANKS WITH WALLS SPANNING VERTICALLY

A SECTION through a long narrow tank or flume with the walls spanning vertically and the roof spanning from wall to wall is shown in *Fig.* 13.1. A controversial point is the net effective upward loading on the floor. The self-weight of the floor and the weight of the water standing on it may be assumed as passing directly to the ground, thus causing no moment in the floor slab, but the weight of the roof and walls must be distributed in some manner under the bottom slab. On comparatively soft ground the designer should assume that it is distributed uniformly across the width of the tank. On a rock foundation it will concentrate directly under the walls and cause little or no bending in the floor. For intermediate cases the distribution shown in *Fig.* 13.1 may be assumed. The free moment under this loading is $0{\cdot}083WL$ and the fixed-ended moment is $0{\cdot}0625WL$.

In *Fig.* 13.2 a preliminary design for a long narrow tank 18 ft. wide and 15 ft.

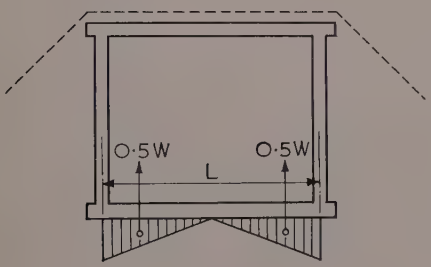


Fig. 13.1.

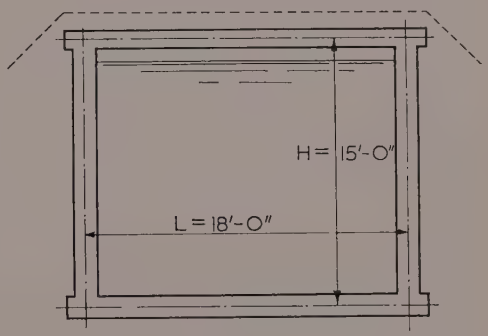


Fig. 13.2.

in height, measured on the centre-lines is shown. Before the moments can be calculated the thickness of the members must be known.

The roof carries 1 ft. of earth and will be about 1 ft. in thickness. With a little imposed load, say 300 lb. per sq. ft.,

Approximate $M = \frac{1}{12} \times 300 \times 18^2 \times 12 = 97{,}000$ lb.-in.

With a resistance moment of $60bd^2$, $d = 11{\cdot}62$ in.; say 12 in. overall.

Total water pressure say $\frac{1}{2} \times 62{\cdot}5 \times 14^2 = 6{,}130$ lb.

M, say $\frac{1}{10}WH = \frac{1}{10} \times 6{,}130 \times 15 \times 12 = 110{,}000$ lb.-in.

With a resistance moment of $60bd^2$, $d = 12{\cdot}37$ in.; say 14 in. overall.

Net upward load on floor:

Roof, say:	20 ft. × 300 =	6,000 lb.
Walls:	2 × 14 ft. × 175 =	4,900 lb.
		10,900 lb.

Say 10,000 lb. on a width of 18 ft.

M = say $\frac{1}{16} \times 10{,}000 \times 18 \times 12 = 135{,}000$ lb.-in.

With a resistance moment of $60bd^2$, $d = 13{\cdot}7$ in.; say 14 in. overall.
The water level is about 1 ft. below the centre-line of the roof slab and this can be dealt with by the method in *Fig.* 12.1.
Outward pressure $= \frac{1}{2} \times 62{\cdot}5 \times 15^2 = 7{,}030$ lb.
Inward pressure $= 1 \times 62{\cdot}5 = 62{\cdot}5$ lb. per foot of height.
With such a small tank temperature movement may be ignored and the rotations of the joints are as shown in *Fig.* 13.3. By symmetry $\theta_3 = -\theta_1$ and $\theta_4 = -\theta_2$.

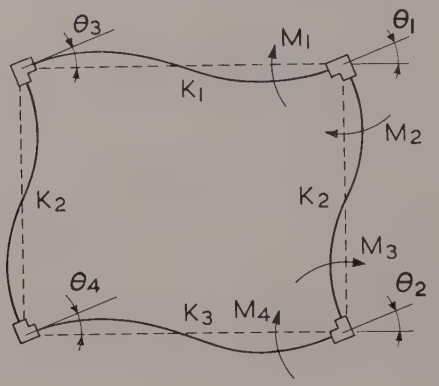


Fig. 13.3.

$$M_{1F} = -\frac{300 \times 18^2}{12} = -8{,}100 \text{ lb.-ft.}$$

$$M_{2F} = -\frac{7{,}030 \times 15}{15} + \frac{62{\cdot}5 \times 15^2}{12} = -5{,}860 \text{ lb.-ft.}$$

$$M_{3F} = +\frac{7{,}030 \times 15}{10} - \frac{62{\cdot}5 \times 15^2}{12} = +9{,}380 \text{ lb.-ft.}$$

$$M_{4F} = +\frac{10{,}000 \times 18}{16} = +11{,}250 \text{ lb.-ft.}$$

All these values are per foot run of tank.
The relative values of K_1, K_2 and K_3 vary inversely as the length and directly as the cube of the thickness, that is $K_1 : K_2 : K_3$ as $\frac{12^3}{18} : \frac{14^3}{15} : \frac{14^3}{18}$ as 1·0 : 1·91 : 1·59.
Tabulating:

	M_1	M_2	M_3	M_4
Joints Fixed	−8,100	−5,860	+9,380	+11,250
Due to θ_1	$4EK_1\theta_1$	$4EK_2\theta_1$	$2EK_2\theta_1$	—
,, ,, θ_2	—	$2EK_2\theta_2$	$4EK_2\theta_2$	$4EK_3\theta_2$
$\theta_3 = -\theta_1$	$-2EK_1\theta_1$	—	—	—
$\theta_4 = -\theta_2$	—	—	—	$-2EK_3\theta_2$

Since $M_1 + M_2 = 0$ and $M_3 + M_4 = 0$, two simple equations for $E\theta_1$ and $E\theta_2$ are obtained.

Solving by slide rule: $M_1 = -2{,}980$ lb.-ft.; $M_2 = +2{,}990$ lb.-ft.; $M_3 = -2{,}280$ lb.-ft.; and $M_4 = +2{,}310$ lb.-ft.

This gives the bending-moment diagram shown on the right-hand side of *Fig.* 13.4. A similar calculation for the empty tank subjected to external earth pressure gives the moment diagram on the left-hand side of *Fig.* 13.4.

In addition to these moments the water pressure will impose a direct tension of about 2,000 lb. on the roof and 4,000 lb. on the floor slab.

The maximum moment in the roof is 9,000 lb.-ft. = 108,000 lb.-in. With a resistance moment of $60bd^2$, $d = 12{\cdot}25$ in. Taking into consideration the direct tension of 2,000 lb., the thickness of the roof slab will have to be increased to 13 in.

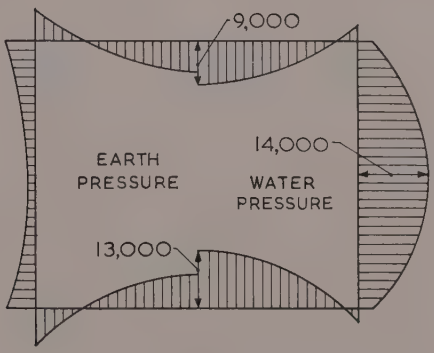


Fig. 13.4.

The maximum moment in the wall is 14,000 lb.-ft., but here tension occurs on the outside face. With a resistance moment of $250bd_1^2$, $d_1 = 7{\cdot}5$ in. only and a 10-in. slab might be sufficient. The maximum moment in the floor is 13,000 lb.-ft. With a resistance moment of $60bd^2$, $d = 14{\cdot}73$ in. and the thickness of the floor should be increased to 15 in.

In practice, too, the inside dimensions, and not the centre-line dimensions, are fixed and changes in thickness mean changes in centre-line dimensions. Changes in thickness also involve changes in the relative values of K_1, K_2 and K_3, with corresponding redistribution of moments and it might be as well to keep to a thickness of 14 in. for the walls to provide extra stiffness.

Once the first preliminary design has been worked out, all these changes are merely matters of arithmetical adjustment common to the design of all statically-indeterminate structures.

Culverts

Culverts are a species of long, narrow tank. The floors are often dished and the roofs sometimes provided with splays. An analysis of the moments in such structures may be found in the writer's ' Frame Analysis'.

Suspended Flumes

The transverse bending moments in an elevated box flume of the type in *Fig.* 13.5, in which the walls act as deep girders, are easily calculated as far as the vertical

loading is concerned, being similar to those in *Figs.* 13.3 and 13·4 except that the effective load on the floor is downwards instead of upwards.

The transverse bending caused by wind loading is more complicated as there are

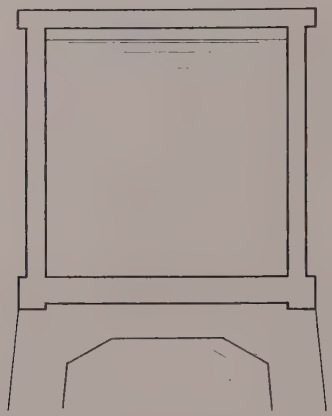


Fig. 13.5.

four different joint rotations together with a lateral displacement of the roof relative to the floor, giving a total of five unknowns. The fifth equation is found from calculating the shear in the vertical walls and equating this to the horizontal wind pressure.

The design of flumes with curved bottoms such as the ' hydraulic catenary ' is outside the scope of this work.

The preliminary designs for a wall of a narrow tank spanning vertically are shown in *Fig.* 13.6. The roof will be assumed to be so small or so well insulated

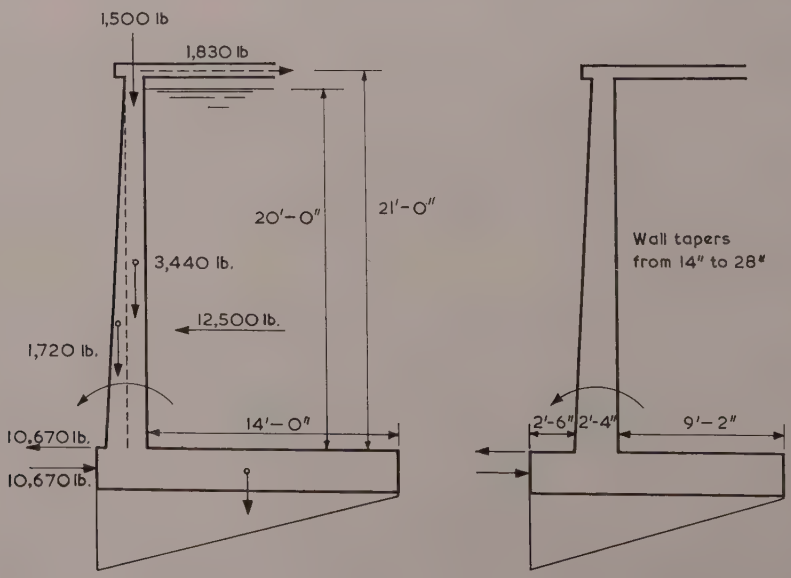


Fig. 13.6.

that its thermal expansion and contraction can be neglected. Practice shows that this is safe for small tanks (say less than 50 ft. wide). The wall is assumed to be fixed in position and direction where it joins the floor and freely supported by the roof slab. It carries 20 ft. of water but is 21 ft. high measured to the centre-line of the roof. In order to use standard moment coefficients this loading may be taken as equivalent to a wall carrying 21 ft. of water on one side and a uniform load of 62·5 lb. per sq. ft. on the other.

Using a wall that tapers in the ratio of 2 to 1,

$$\begin{aligned} M \text{ at base} &= 0{\cdot}085 \times (21 \times 62{\cdot}5) \times 21^2 = 49{,}300 \text{ lb.-ft.} \\ &\quad - 0{\cdot}1668 \times 62{\cdot}5 \times 21^2 = \underline{\ 4{,}600 \text{ lb.-ft.}} \\ &\qquad\qquad\qquad\qquad\qquad\quad 44{,}700 \text{ lb.-ft.} \end{aligned}$$

The full cantilever moment if the wall were not attached to the roof would be

$$0{\cdot}167 \times 62{\cdot}5 \times 20^3 = 83{,}200 \text{ lb.-ft.}$$

$$\text{Top reaction} = \frac{83{,}200 - 44{,}700}{21} = 1{,}830 \text{ lb.}$$

Total water pressure $= 0{\cdot}5 \times 62{\cdot}5 \times 20^2 = 12{,}500$ lb.
Bottom shear $= 12{,}500 - 1{,}830 = 10{,}670$ lb.
For a resistance moment of $60bd^2$, the thickness at the bottom of the wall is

$$\sqrt{\frac{44{,}700 \times 12}{60 \times 12}} = 27{\cdot}3, \text{ say } 28 \text{ in.}$$

Making the base slab also 28 in. in thickness and assuming that the bottom shear of 10,670 lb. is taken by counterpressure on the face of the toe, the width of base necessary to keep the resultant inside the middle third has been found by trial and error, firstly, on the left of the figure, with the base projecting only 6 in. to form a 6-in. toe, and secondly, on the right of the figure, with a 2-ft. 6-in. toe. The vertical loading from the roof coming on the top of the wall is 1,500 lb. per foot run. The stability of the base of the right-hand detail is shown in *Fig.* 13.7. Taking moments about point X,

1,720	× 3·28	= 5,640	lb.-ft.
4,940	× 4·25	= 21,000	,, ,,
4,700	× 7·0	= 32,900	,, ,,
11,450	× 9·42	= 108,000	,, ,,
22,810	lb.	167,540	lb.-ft.
less	44,700		
10,670	× 1·17 = 12,400	57,100	,, ,,
		110,440	lb.-ft.

$$\text{Horizontal distance of } R \text{ from point X} = \frac{110{,}440}{22{,}810} = 4{\cdot}84 \text{ ft.}$$

R is $7{\cdot}0 - 4{\cdot}84 = 2{\cdot}16$ ft. out of centre.

$$\text{Pressure on ground} = \frac{22{,}810}{14} \pm \frac{22{,}810 \times 2{\cdot}16 \times 6}{14^2} = 1{,}630 \pm 1{,}510 = 3{,}140 \text{ to } 120 \text{ lb. per sq. ft.}$$

Of the total upward ground pressure of 22,810 lb., 10,200 lb. acts under the tail at a distance of 3·22 ft. from Section *a–a* in *Fig.* 13.7. The unit downward loading on the tail is shown overleaf.

Water: $20 \times 62{\cdot}5 = 1{,}250$ lb.
Self-weight: $28 \times 12 = 336$ lb.
$1{,}586$ lb.

M on Section a–$a = \frac{1}{2} \times 1{,}586 \times 9{\cdot}17^2 = 66{,}700$ lb.-ft.
less $10{,}200 \times 3{\cdot}22 = 32{,}800$ „ „
$33{,}900$ lb.-ft.

This is less than the moment of 44,700 lb.-ft. at the bottom of the wall slab and a thickness of 2 ft. 4 in. is ample for the base slab. This is generally true of all walls that have a reasonably large projecting toe; it is not true for walls that have very small toes. The corresponding moment for the wall in the left-hand side of *Fig.* 13.6 is 50,800 lb.-ft. and for a resistance moment of $60bd^2$ a thickness of 2 ft. 4 in. is not enough. If the horizontal shear of 10,670 lb. cannot be taken by counter-pressure on the front of the toe and has to be taken in shear friction on the underside of the base slab, this moment would be increased to 62,800 lb.-ft.

When comparing the cost of a wall of this type with the cost of a free-standing

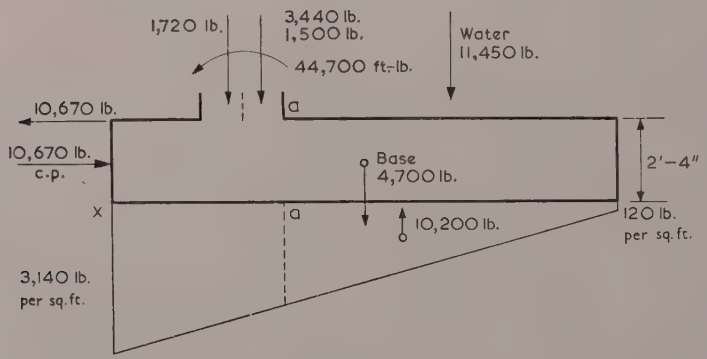


Fig. 13.7.

cantilevered wall, the designer must remember to include the cost of the extra steel (and possibly extra concrete) in the roof to carry the tension of 1,830 lb. per foot run across the roof from wall to wall.

If the tank is banked round after it has been filled and tested, and if the earth pressure is 20 lb. per sq. ft. per foot of depth, the ground-level being 1 ft. 4 in. above the centre-line of the roof, the earth pressure on the wall consists of a triangular load varying from zero to 420 lb. per sq. ft., plus a uniform load of 26·6 lb. per sq. ft. When the tank is empty the moment at the bottom of the wall is

$0{\cdot}085 \times 420 \times 21^2 = 15{,}800$ lb.-ft.
$+ 0{\cdot}1668 \times 26{\cdot}6 \times 21^2 = 1{,}950$ „ „
$17{,}750$ lb.-ft.

In *Fig.* 13.8 an alternative design to that in *Fig.* 13.6 is shown, with vertical beams tied into the floor and roof and the wall slab spanning horizontally between these vertical beams. The design of the wall slab will be similar to the wall slab in a cantilevered counterfort wall, *q.v.* The top end of each beam is tied into the roof at a point 1 ft. above the water level, the bottom end being tied 8 in. below the top of the floor, giving a total vertical span of 21 ft. 8 in. If the beams are spaced

at 10-ft. centres, the total water pressure carried by each is $0{\cdot}5 \times 62{\cdot}5 \times 20^2 \times 10$ or 125,000 lb. The top reaction is 42,300 lb. and the bottom reaction is 82,700 lb. The maximum moment occurs 11·63 ft. below water level and is 4,450,000 lb.-in. Using a four-bar arrangement with pairs of $\frac{3}{8}$-in. diam. stirrups, with $1\frac{1}{2}$-in. concrete cover outside the stirrups and 2 in. between bars, as shown in *Fig.* 13.9, a beam about 15 in. wide will be required. Taking the bottom shear at 200 lb. per sq. in. on the lever-arm area, lever arm required $= \dfrac{82{,}700}{200 \times 15} = 27{\cdot}6$ in.; say 36 in. deep overall.

A_{st} at 18,000 lb. per sq. in. $= \dfrac{4{,}450{,}000}{18{,}000 \times 27{\cdot}6} = 8{\cdot}97$ sq. in.; say eight $1\frac{1}{4}$-in. diam. bars.

According to CP.2007:1960 the stress ' in shear reinforcement ' is limited to 12,000 lb.

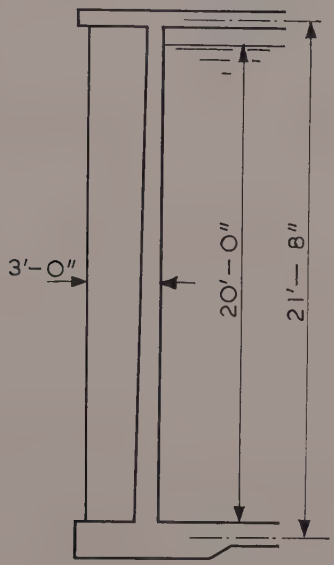


Fig. 13.8.

per sq. in. This presumably refers to links and stirrups but whether it is intended also to apply to inclined bars, the middle sections of which are stressed to 18,000 lb. per sq. in. in bending is not clear.

A detail for the bottom end of the beam is shown in *Fig.* 13.9. A reasonable working stress for this detail would be 16,000 lb. per sq. in. in both inclined bars and stirrups. The minimum spacing of links, allowing 2 in. clear between pairs, is $2\frac{3}{4}$ in.

Four $1\frac{1}{4}$-in. diam. bars at 30 deg., at 16,000 lb. per sq. in. = 39,300 lb.

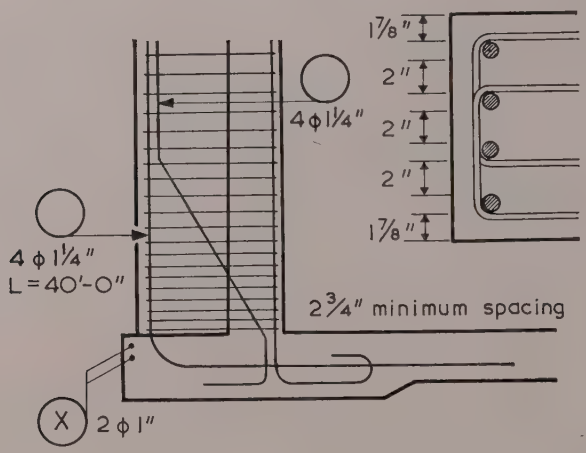
Pairs of $\frac{3}{8}$-in. diam. stirrups at $2\frac{3}{4}$-in. centres $= \dfrac{0{\cdot}4416 \times 27{\cdot}6 \times 16{,}000}{2{\cdot}75} =$ 71,000 lb.

110,300 lb.

Reducing the stress in the steel from 16,000 to 12,000 lb. per sq. in. reduces the calculated shear resistance from 110,300 to 82,900 lb.

The bottom end of the beam must be properly anchored. On solid rock no problem arises. On other types of soil the tying-in bars must pick up a sufficient

area of floor to keep the shear-friction stress under the floor slab within safe limits. At a safe figure of 1,500 lb. per sq. ft., a reaction of 82,700 lb. needs an area of 10 ft. by 5 ft. 6 in. and the arrangement in *Fig.* 13.9 would be ample. On a weaker site the reinforcement in the floor slab must be sufficiently strong to carry part of



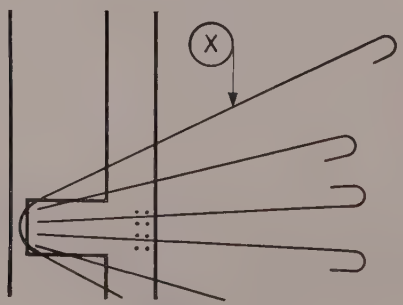


Fig. 13.9.

the reaction further in towards the centre of the floor. The tying-in steel shown in *Fig.* 13.9 consists of four 1¼-in. diam. bars running more or less straight into the floor and two 1-in. diam. bars marked X at about 30 deg. The resolved tension is, say,

$$4 \times 1{\cdot}23 \times 12{,}000 = 59{,}000 \text{ lb.}$$
$$4 \times 0{\cdot}785 \times 12{,}000 \times 0{\cdot}75 = 28{,}300 \text{ lb.}$$
$$87{,}300 \text{ lb.}$$

The difficulties with this arrangement lie in the construction. The reinforcement skeletons for the beams must be assembled, erected, and held firmly and accurately in place before the floor is concreted. The shuttering for the wall slab and beams should be so designed that there is an interval of not more than 3 or 4 hours between concreting the floor and concreting the lower part of the wall. Ideally, the floor and wall up to the point of zero shear in the beams should be concreted in one

continuous operation, but this would entail shuttering a considerable area of the top surface of the floor, a practice which is not usual in this country. No normal specification would allow a straight construction joint in a floor beam at the face of the column with a total shear of 82,700 lb. in the beam, as is the case here.

Designers who have little experience of practical construction should consider this point carefully before adopting this form of construction. If delays to concreting are unavoidable the cross-section of the beam may be increased, to reduce the shear stress on the bottom construction joint to about 100 lb. per sq. in., or the designer may revert to the plain slab design in *Fig.* 13.6, where the shear force of 10,670 lb. comes on a lever-arm area of 12 in. by 22·5 in., giving a shearing stress of less than 40 lb. per sq. in.

Mesh-panel Walls

Unlike mesh-panel roofs, where standard practice has clearly indicated the moment coefficients that should be used, mesh-panel walls have not been used sufficiently often to establish any standard practice. There is, in fact, one very good reason why a single standard moment coefficient cannot be used and that is because there is no standard ' edge condition ' at the top edge or bottom edge of the panel.

In *Fig.* 13.10 the elevation of a mesh-panel wall of height h and span l carrying

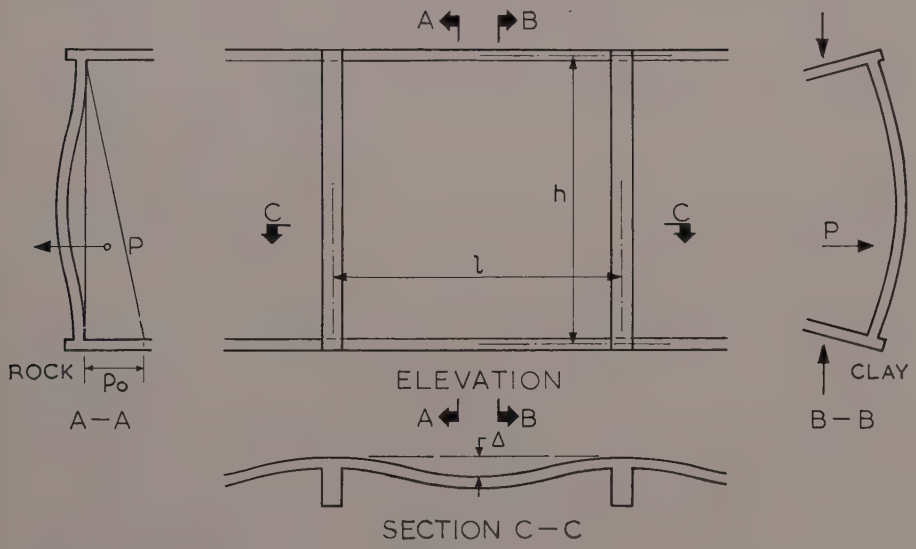


Fig. 13.10.

a triangular load P, where $P = \frac{1}{2}p_0hl$, is shown, the width l lying between $0{\cdot}9h$ and $1{\cdot}1h$. The edge conditions along the vertical edges are equivalent to their being fixed in position and direction since the panels are, in practice, invariably made all the same width. Section C–C shows the type of horizontal section these conditions will produce, although the value of Δ will not be known until the edge conditions along the top and bottom edges of the panel are determined. Section A–A shows a vertical section, assuming that the tank has a stiff roof slab carrying very little load (an unlikely combination) and that it sits on an unyielding rock

foundation. Under these conditions the panel approximates to the condition of being fixed in position and direction on all four edges and could be designed for unit moments of $+\frac{1}{24}P$ and $-\frac{1}{24}P$ at centre and supports. Section B–B shows a vertical section, assuming that the roof slab is flexible and heavily loaded and that the load on the foundation tends to spread towards the centre of the tank. The twisting of the top and bottom edges of the panel increases the central deflection

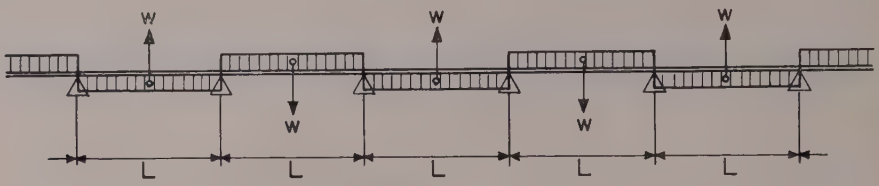


Fig. 13.11.

Δ and affects the moments in the panel. If the moments imposed on the top and bottom edges by the roof and floor are approximately equal to $+\frac{1}{24}P$, the panel may be designed for $+\frac{1}{16}P$ and $-\frac{1}{16}P$. If the twisting at top and bottom is more severe than this, then the design should be modified or, possibly, some other form of construction should be adopted.

The condition at the top and bottom of the wall panel in Section B–B in *Fig.* 13.10

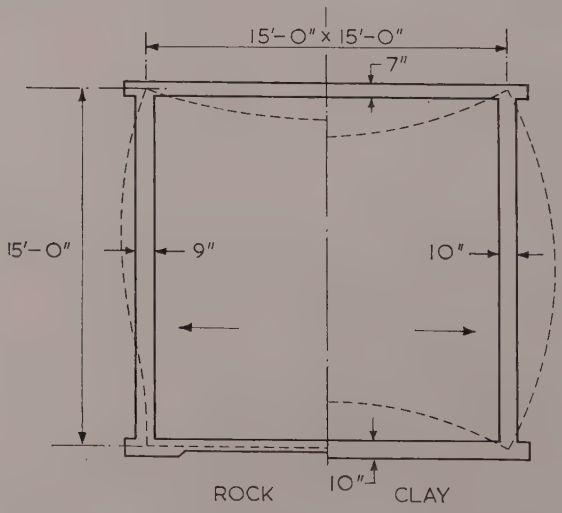


Fig. 13.12.

is not ' continuous ' in the accepted sense of the word, for the loading on the roof is downward, the loading on the wall is outward and the loading on the floor is upward. The roof, wall and floor are only ' continuous ' in the sense that the continuous beam shown in *Fig.* 13.11 is continuous. In such a beam under such loading, the moment at the points of support is zero, the midspan moments being $\frac{1}{8}wL^2$; each span, in effect, behaving like a freely-supported beam.

A preliminary design for a tank measuring 15 ft. by 15 ft. by 15 ft. is shown in *Fig.* 13.12. The floor, walls and roof may be designed as mesh-panels.

Before the moment coefficients to be used in designing the walls can be decided, the loading on the roof and on the floor must be considered.

Roof slab. Load: self-weight: 84 lb. per sq. ft.
imposed load: 25 lb. per sq. ft.
say 110 lb. per sq. ft.

The moments in the roof slab are almost invariably modified by the introduction of at least one access manhole. Provided this is small (say 18 in. by 18 in.), the roof slab may be designed to resist moments of $\pm \frac{1}{20}W$, or for a larger manhole, say $\pm \frac{1}{18}W$, where W is the total load on the roof panel. For openings larger than 24 in. by 24 in. the idea of mesh-panel construction should be abandoned, the roof slab being designed as a simple span.

With $1\frac{1}{2}$-in. concrete cover a 6-in. slab is rather thin for mesh-panel work and a minimum thickness of 7 in. has been taken here.

$$W = 15 \times 15 \times 110 = 24{,}800 \text{ lb.}$$

$$M = \text{say } \pm \frac{W}{18} = \pm \frac{24{,}800 \times 12}{18} = 16{,}500 \text{ lb.-in. per foot width.}$$

Minimum d required for a resistance moment of $60bd^2 = 4{\cdot}78$ in.

$$A_{st} = \frac{16{,}500}{12{,}000 \times 0{\cdot}86 \times 5} = 0{\cdot}32 \text{ sq. in.; say } \tfrac{1}{2}\text{-in. diam. bars at 7-in. centres.}$$

Total pressure on wall panel $= \frac{1}{2} \times 15 \times 15^2 \times 62{\cdot}5 = 105{,}500$ lb.

$$\text{On a rock base, unit } M = \text{say } \pm \frac{P}{24} = \pm \frac{105{,}500 \times 12}{24} = 52{,}800 \text{ lb.-in. per foot width.}$$

Minimum d required for resistance moment of $60bd^2 = 8{\cdot}56$, say 9 in.

$$A_{st} \text{ near outside face} = \frac{52{,}800}{18{,}000 \times 0{\cdot}86 \times 7} = 0{\cdot}488 \text{ sq. in.}$$

A_{st} near inside face, at 12,000 lb. per sq. in. $= 0{\cdot}731$ sq. in.

The reinforcement is shown in *Fig.* 13.13. In *Fig.* 13.13 bars C and D are $\frac{5}{8}$-in. diam. bars at 7-in. centres. Corner bars B and H are $\frac{5}{8}$-in. diam. bars at 5-in. centres. The reversed moment at the top edge of the wall panel will be small (see *Fig.* 13.4) and bars G are $\frac{1}{2}$-in. diam. bars at 6-in. centres. Bars E and F pick up the ends of bars C, which are too long to be turned over at the top. They need not be more than one-half of the area of bars C and $\frac{1}{2}$-in. diam. bars at 7-in. centres are ample. Bars A are 0·3 per cent. of 9 in. by 12 in.

On a rock base the centre of the floor slab may be thinned out as shown in *Fig.* 13.12. This is generally not worth while for a small tank. In *Fig.* 13.13 a floor of uniform thickness is shown.

For the tank on a clay foundation a larger moment coefficient must be used for the wall panel, something between $\frac{1}{24}P$ and $\frac{1}{16}P$. Subject to a final check when all the moments have been worked out, M may be assumed as $\pm \frac{1}{20}P$ for a preliminary design.

$$\pm \frac{P}{20} = \pm \frac{105{,}500 \times 12}{20} = 63{,}300 \text{ lb.-in. per foot width.}$$

Minimum d required for resistance moment of $60bd^2 = 9{\cdot}36$, say 10 in.

$$A_{st} \text{ near outside face} = \frac{63{,}300}{18{,}000 \times 0{\cdot}86 \times 8} = 0{\cdot}512 \text{ sq. in.}$$

A_{st} near inside face, at 12,000 lb. per sq. in. $= 0{\cdot}766$ sq. in.

Bars L are $\frac{5}{8}$-in. diam. bars at $4\frac{1}{2}$-in. centres, while bars N are $\frac{5}{8}$-in. diam. bars at 7-in. centres.

Assuming that the water pressure on the floor and the self-weight of the floor slab pass directly to the ground, the effective upward load on the floor slab is

Roof: say $17 \times 17 \times 100 =$ 28,900 lb.
Walls: $4 \times 15 \times 15 \times 125 =$ 112,500 lb.
141,400 lb.

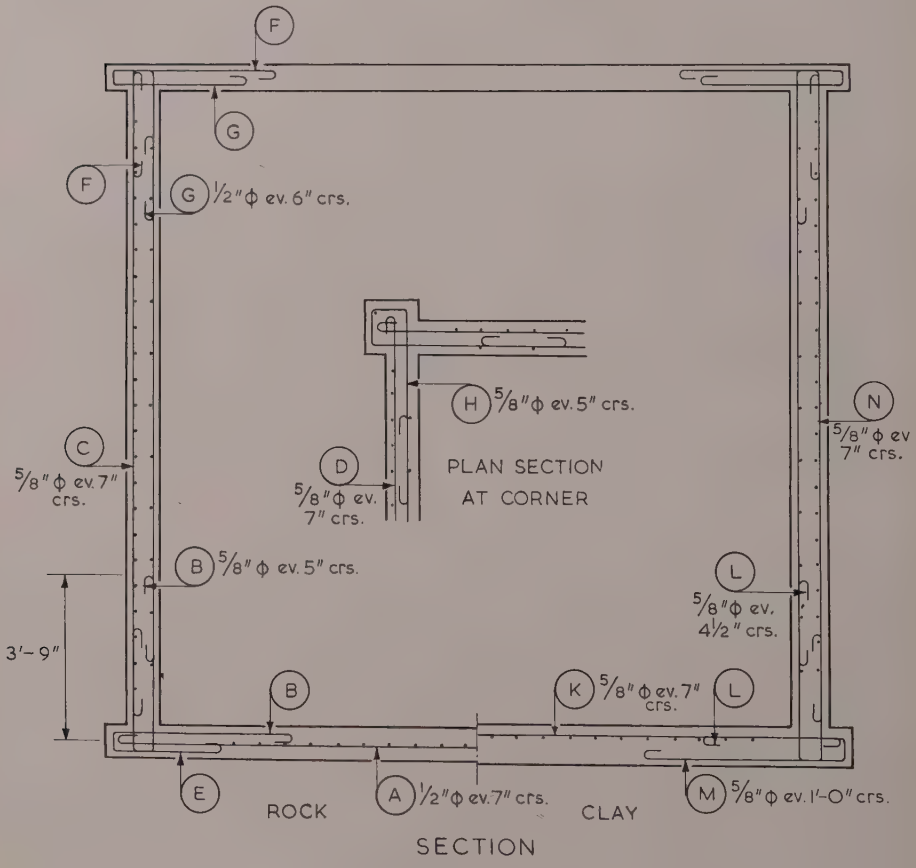


Fig. 13.13.

Assuming that this load is evenly distributed over an area of 17 ft. by 17 ft., this is a load of 490 lb. per sq. ft.
Total effective upward load on floor slab, $W = 15 \times 15 \times 490 = 110{,}000$ lb.

This load will be effective whether the tank is full or empty. Filling the tank will decrease the edge moments but will increase the central moment. The floor slab may be designed for $+\frac{1}{30}W$ and $-\frac{1}{36}W$.

$$\text{Central } M = \frac{110{,}000 \times 12}{30} = 44{,}000 \text{ lb.-in. per foot width.}$$

$$A_{st} = \frac{44{,}000}{12{,}000 \times 0{\cdot}86 \times 8} = 0{\cdot}533 \text{ sq. in.}$$

$$\text{Edge } M = \frac{110{,}000 \times 12}{36} = 36{,}600 \text{ lb.-in. per foot width.}$$

$$A_{st} = \frac{36{,}600}{18{,}000 \times 0{\cdot}86 \times 8} = 0{\cdot}296 \text{ sq. in.}$$

The top detail of the wall in the right-hand side of *Fig.* 13.13 will be similar to the detail in the left-hand side. The bars in the roof slab are $\frac{1}{2}$-in. diam. bars at 7-in. centres or, say, 24 bars in each direction, those bars which would normally cross the manhole being placed at each side of it. This completes all the work necessary for a preliminary design.

For a final design the centre-line dimensions are fixed from the given internal dimensions. The total pressure on the wall slab will generally be less than the figure already taken, since the top water level is usually several inches below the roof.

On a clay foundation the calculated edge moment in the roof slab is 16,500 lb.-in. and the calculated edge moment in the floor slab is 36,600 lb.-in., while the calculated moment in the wall panel is $\pm$ 63,300 lb.-in. In view of the positive edge moments it would be safer to increase the central moment in the wall panel to $\frac{1}{18}P$.

As in all mesh-panel work the designer should start with full-scale bar-spacing sections to make sure that each layer of bars has its own definite location and cannot interfere with any other layer. The effective depth may be taken as the average effective depth of the two layers. In a 9-in. wall reinforced with $\frac{5}{8}$-in. diam. bars in both directions and having $1\frac{1}{2}$-in. concrete cover, the effective depth may be taken as $6\frac{7}{8}$ in. If the requirements of CP.2007:1960 are being observed, the thickness of the walls will be not less than 9 in., in order to take advantage of the higher allowable stresses.

Looking at *Fig.* 13.13 from a common-sense viewpoint, the author would reduce the spacing of bars C and D to 6 in. and increase the spacing of bars B and H to 6 in.

In the final design notice must also be taken of the direct tension due to the bursting pressure. The reactions round the edge of a mesh panel of uniform elastic material may be calculated for various edge conditions, but the edge reactions of a mesh-panel of reinforced concrete under the conditions of support in *Fig.* 13.12 are unknown except, of course, for the fact that the centre of gravity of the combined reactions must coincide with the centre of gravity of the loading. For a square panel L ft. by L ft. carrying a triangular load which varies from zero at the top edge to pL at the bottom edge, as a first approximation a uniform reaction of $0{\cdot}0625pL^2$ may be assumed along the top edge, $0{\cdot}1875pL^2$ along the bottom edge, and side reactions varying uniformly from $0{\cdot}0625pL^2$ at the top to $0{\cdot}1875pL^2$ at the bottom. If $L = 15$ ft. and $p = 62{\cdot}5$ lb. per cu. ft., reaction along the top edge = 880 lb. per foot, and reaction along the bottom edge = 2,640 lb. per foot. A tension of 880 lb. per foot on a 7-in. slab causes a tension of 10·5 lb. per sq. in. in the concrete. Taking one-half of the tension on the tensile steel at 12,000 lb. per sq. in., an additional area of 0·037 sq. in. is required. Halfway down the wall the horizontal tension is 1,760 lb. per foot or 16·3 lb. per sq. in. on a 9-in. slab. Half this tension, at 18,000 lb. per sq. in. requires an additional area of steel of 0·098 sq. in.

Mesh Panel, Freely Supported

A solution to the problem of a rectangular slab freely supported at all four edges and subjected to triangular loading, derived by Navier's method, is given in ' Handbuch für Eisenbetonbau'.

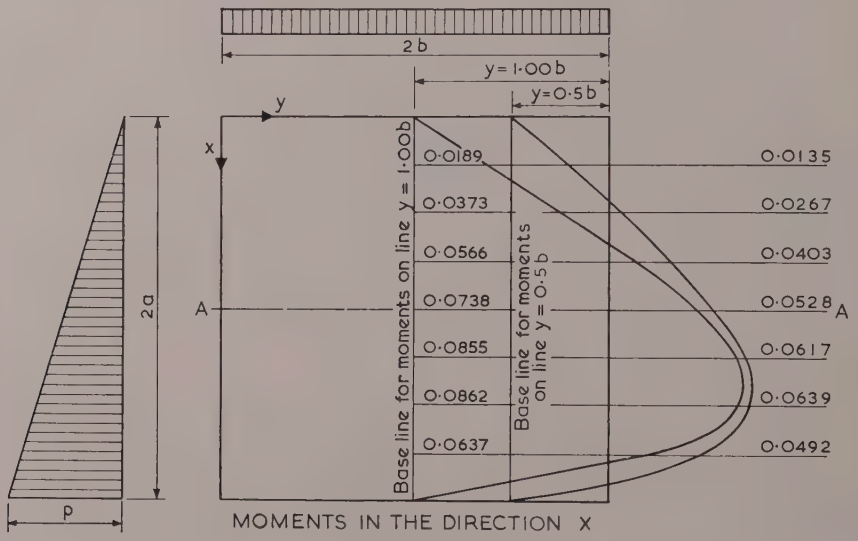


Fig. 13.14.

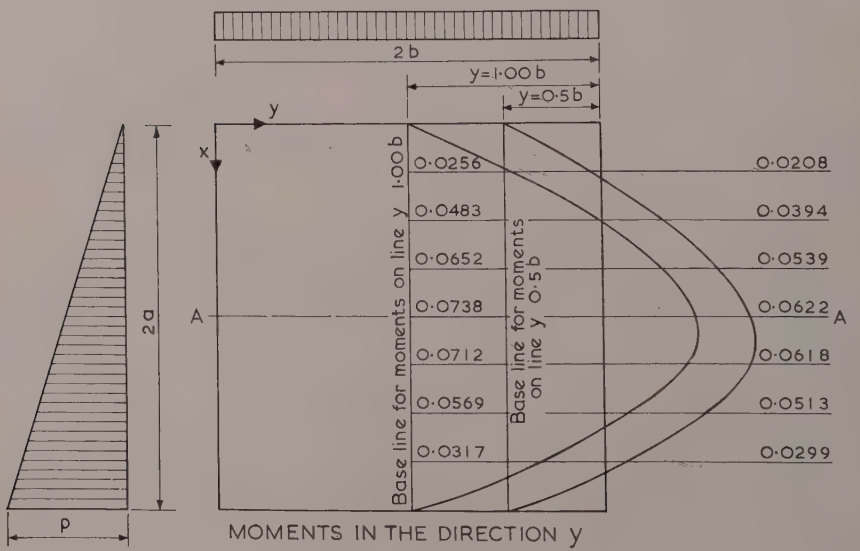


Fig. 13.15.

For a square slab the coefficients of the moments at different points in the two directions are shown in *Figs.* 13.14 and 13.15. It should be noted that $M = pa^2 \times$ coefficient, in which a is one-half of the length of the side of the square and p is the maximum intensity of the triangularly-distributed load.

For freely-supported rectangular slabs under triangular loading, the coefficients

for the moments on the horizontal centre-line (AA) for various ratios $k = b/a$ are shown in the following table.

MOMENTS IN DIRECTION x.

$\frac{y}{b}$	$k = 0.25$	0.33	0.50	0.67	1.0	1.5	2.0	3.0	4.0	∞
0.50	0.00004	0.0009	0.0063	0.0176	0.0528	0.1068	0.1474	0.1957	0.2212	0.5
1.00	0.00006	0.0013	0.0088	0.0248	0.0738	0.1453	0.1930	0.2344	0.2459	0.25

MOMENTS IN DIRECTION y.

$\frac{y}{b}$	$k = 0.25$	0.33	0.50	0.67	1.0	1.5	2.0	3.0	4.0	∞
0.50	0.0115	0.0196	0.0368	0.0506	0.0622	0.0576	0.0460	0.0296	0.0183	0
1.00	0.0154	0.0260	0.0483	0.0646	0.0738	0.0557	0.0353	0.0113	0.0010	0

In each case the moment $(M) = pa^2 \times$ coefficient.

The moments are such that tension is produced at the outer face of the panel if due to internal outward pressure, and compression if due to external inward pressure.

Mesh Panel Supported on Three Edges

Normally a mesh-panel wall slab is supported along its upper edge either by the roof slab or by a top beam spanning from counterfort to counterfort. Since the maximum loading and maximum moments occur in the lower part of the panel it is possible, although not always desirable, to omit the beam (or other supporting member) at the top edge of the panel. An analysis of this case, assuming that the panel is fixed in position and direction along its bottom edge and along both vertical edges but completely unsupported along its top edge, was given by J. F. Büchi in ' Beton und Eisen ' for April 20, 1938, and was noted in ' Concrete and Constructional Engineering ' for July 1951. The curves in *Fig.* 13.16 are based on this method and give the bending-moment coefficients for various ratios of length to height of such walls.

In accordance with this method the maximum bending moment M_H at the vertical edges of a panel is $\frac{1}{12}\alpha p_0 l^2$ on an imaginary horizontal strip of unit width, if p_0 is the pressure at the bottom of the wall and l is the length of the panel. This bending moment occurs on a strip at a height of one-quarter to two-thirds of the depth h of the tank; the greater the length compared with the depth the higher the point of maximum bending moment.

The bending moment, horizontally, at the middle of the wall is about half that at the corners at the same height.

The maximum bending moment M_v on a vertical strip of unit width is $\frac{1}{6}\beta p_0 h^2$ and occurs at the middle of the bottom edge of the panel.

The coefficients α and β depend on the ratio m $(= l/h)$ and are given by $\alpha = \left(1 - \frac{m}{6{\cdot}2}\right)^4$ and $\beta = \frac{m - 0{\cdot}3}{3}$.

The curves give values of $\frac{1}{12}\alpha$ $(= \alpha')$ and $\frac{1}{6}\beta$ $(= \beta')$ for various ratios of l/h. This method is not applicable to panels having ratios of l/h of less than one-half or greater than two. A tall narrow panel, that is a panel having a ratio of less than one-half, tends to span entirely horizontally, and the maximum bending moment occurs near the bottom and will approach $\frac{1}{12}p_0l^2$, that is $\alpha = 1$, and $\alpha' = \frac{1}{12}$.

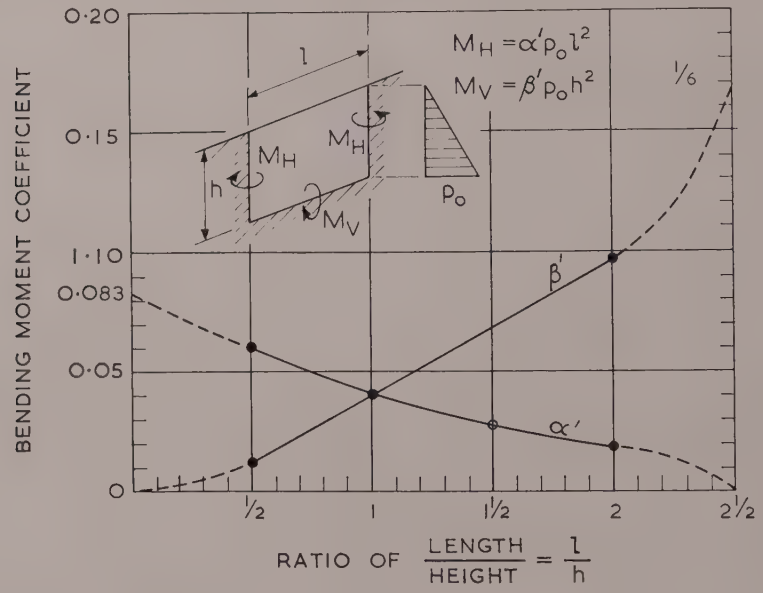


Fig. 13.16.

A long low panel, that is a panel having a ratio of greater than two, tends to act entirely as a vertical cantilever, the maximum bending moment at the bottom being $\frac{1}{6}p_0h^2$, that is $\beta = 1$ and $\beta' = \frac{1}{6}$.

The curves are drawn so that they tend to approach these limiting values of $\alpha' = \frac{1}{12}$ and $\beta' = \frac{1}{6}$ but conform to $\frac{1}{12}\alpha$ and $\frac{1}{6}\beta$ respectively for the intermediate range of $l/h = \frac{1}{2}$ to 2.

It must be noted that this analysis applies only to the case where the bottom edge of the panel is fixed in position and direction. The imposition of a positive bending moment on the bottom edge of the panel will modify the moments in the panel in both directions.

CHAPTER FOURTEEN

USE OF SEGMENTAL ARCHES IN RESERVOIR WALLS

The modern insistence on low tensile stresses in concrete and steel has made the use of the arched type of wall shown in *Fig.* 14.1 more attractive, since the primary stresses are simple direct compressions. If, however, the small arches are too flat or the counterfort is too flexible the resulting ' crown-drop ' effect may cause tensile stresses which exceed the primary compression stress.

When the arch occurs in the centre of a long straight wall the loading on the

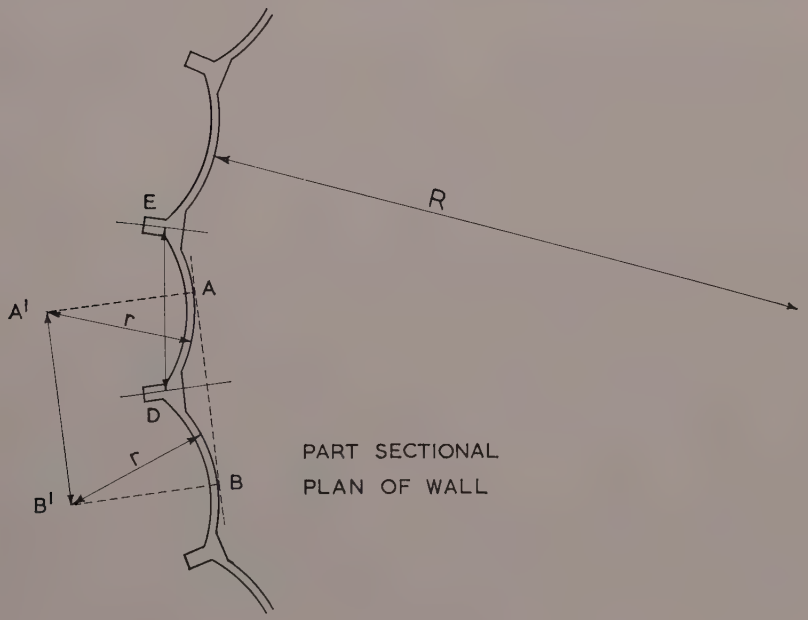


Fig. 14.1.

counterfort (or vertical beam) is clear and the effect of the deflection of the counterfort on the stresses in the arch, acting as an arch, is zero.

When the arch occurs in a circular reservoir the loading on the counterfort is not equal to $0{\cdot}5wH^2 \times$ (spacing of counterforts). Part of a circular reservoir is shown in *Fig.* 14.1, in which the principal radius of the reservoir is R and the radius of the small arches is r. In *Fig.* 14.1 the line AB is drawn tangential to two consecutive arches. The thrust in each arch at points A and B is parallel to the line AB and the total load on each counterfort is $0{\cdot}5wH^2 \times$ (length AB) and not $0{\cdot}5wH^2 \times$ (length ED). By geometry AB = A'B', which is the distance between consecutive arch centres. In *Fig.* 14.2 a corner counterfort in a square reservoir is shown. The load on this counterfort is $0{\cdot}5wH^2 \times$ (length AB) and here again AB = A'B', which is the distance between the arch centres.

First, the case of an arch in a straight wall shown in *Fig.* 14.3 will be considered. Assuming that the main corners of the reservoir are firmly held by stout counterforts (see *Fig.* 14.2) and the bottom floor of the reservoir does not stretch, the span ED will remain constant. The primary stress due to water pressure is a simple compression, say F_c. This state of compression causes a shortening of the arch which

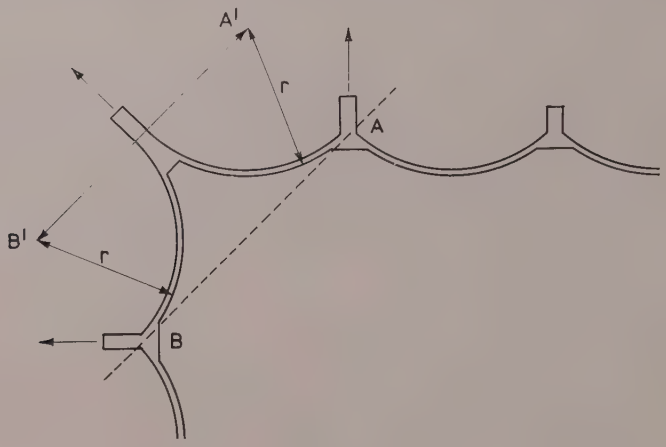


Fig. 14.2.

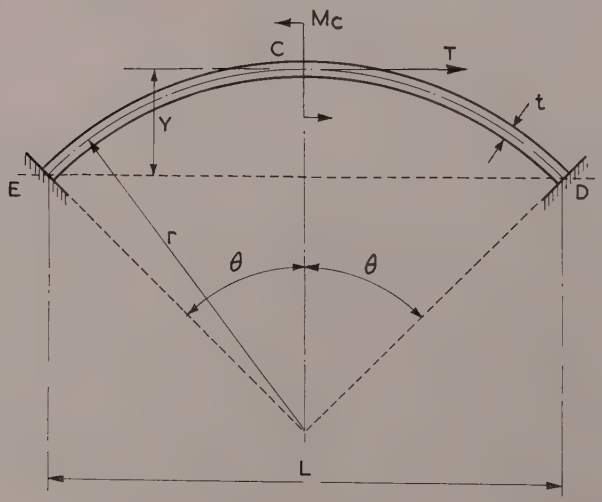


Fig. 14.3.

induces severe crown-drop moments and crown-drop tension. This tension reduces the primary compression stress F_c to a smaller residual compression stress f_c. A uniform stress of f_c would cause a shortening of the arch equal to f_c/E_c. If f_c is 300 lb. per sq. in. and $E_c = 3{,}000{,}000$ lb. per sq. in., the arch will shorten one ten-thousandth of its length. If the crown-drop tension is T lb. per foot height of wall,

$$f_c = F_c - \frac{T}{(\text{area of section})} = F_c - \frac{T}{12t}.$$

Values of M_c, M_E and T for different values of θ are given in *Fig.* 14.4.

If $\theta = 30$ deg., $T = 593 f_c \frac{t^3}{r^2}$ lb. per foot of height,

and $$\frac{T}{(\text{area of section})} = 593 f_c \frac{t^3}{r^2} \times \frac{1}{12t} = 49{\cdot}5 f_c \left(\frac{t}{r}\right)^2 \text{ per sq. in.}$$

If $t = 6$ in. and $r = 120$ in., then $49{\cdot}5 f_c \left(\frac{t}{r}\right)^2 = 0{\cdot}124 f_c$.

For these values of t, r and θ then $f_c = F_c - 0{\cdot}124 f_c$, or $f_c = 0{\cdot}89 F_c$.

If the primary compression stress F_c in an arch with these particular dimensions is 200 lb. per sq. in. the crown-drop tension will reduce this to

$$f_c = 0{\cdot}89 \times 200 = 178 \text{ lb. per sq. in.,}$$

If $t = 6$ in., $r = 120$ in., $\theta = 30$ deg., and the maximum effective water pressure is due to a head of 20 ft.:

Primary compression in arch per foot of height $= 20 \times 62{\cdot}5 \times 10$ ft. $= 12{,}500$ lb.

$$F_c = \frac{12{,}500}{12 \times 6} = 174 \text{ lb. per sq. in.}$$

$$T \text{ (from } \textit{Fig.} \text{ 14.4)} = 593 f_c \frac{t^3}{r^2} \text{ lb. per foot of height}$$

$$= 593 f_c \times \frac{6^3}{120^2} \text{ lb. per foot of height}$$

$$= 593 f_c \times \frac{6^3}{120^2} \times \frac{1}{6 \times 12} \text{ lb. per sq. in.}$$

$$= 0{\cdot}124 f_c \text{ lb. per sq. in.}$$

$$f_c = F_c - 0{\cdot}124 f_c \text{ or } f_c = 0{\cdot}89 F_c$$

$$= 0{\cdot}89 \times 174 = 155 \text{ lb. per sq. in.}$$

$$M_E = 52{\cdot}6 f_c \frac{t^3}{r} \text{ and } M_c = 26{\cdot}8 f_c \frac{t^3}{r} \text{ (}\textit{Fig.}\text{ 14.4).}$$

$$M_E = 52{\cdot}6 \times 155 \times \frac{6^3}{120} = 14{,}700 \text{ lb.-in. per foot of height.}$$

Stresses due to this moment $= \pm 14{,}700 \times \frac{6}{12 \times 6^2} = \pm 204$ lb. per sq. in.

Adding the direct compression of 155 lb. per sq. in., the stress at sections E and D varies from a tension of 49 lb. per sq. in. on the water face to a compression of 359 lb. per sq. in. on the earth face. Similarly at the midspan point C the stresses are $+155 \pm 104$, or $+259$ to $+51$ lb. per sq. in.

It is, of course, permissible to allow a calculated tensile stress of about 300 lb. per sq. in. in the concrete, but if the object of the designer is to eliminate all tension, the adoption of flat arches tends to defeat this primary aim.

If $t = 6$ in., $r = 120$ in., $\theta = 45$ deg., and the maximum effective head of water pressure is 25 ft.:

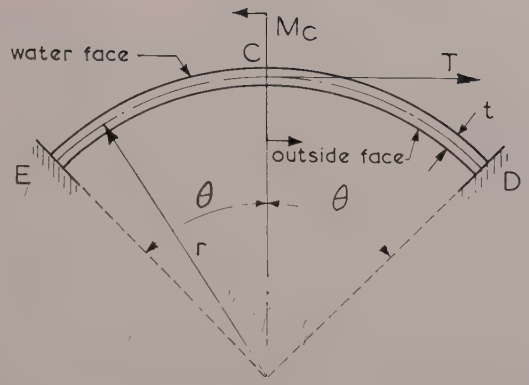
Primary compression in arch per foot of height $= 25 \times 62{\cdot}5 \times 10 = 15{,}600$ lb.

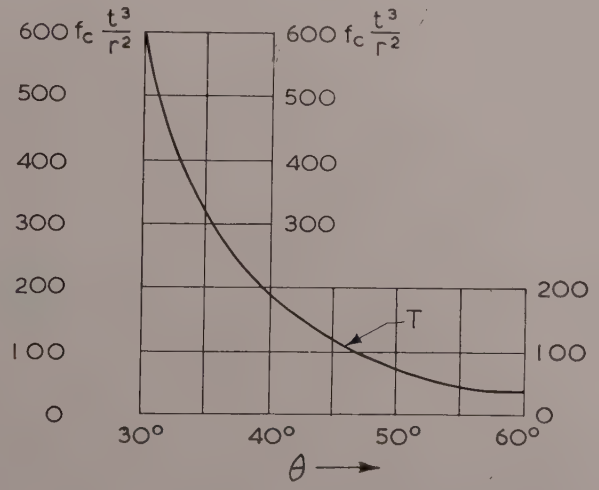
$$F_c = \frac{15{,}600}{12 \times 6} = 217 \text{ lb. per sq. in.}$$

$$T \text{ (from } \textit{Fig.} \text{ 14.4)} = 116{\cdot}5 f_c \frac{t^3}{r^2} \text{ lb. per foot of height.}$$

$$= 116{\cdot}5 f_c \times \frac{6^3}{120^2} \times \frac{1}{6 \times 12} = 0{\cdot}0243 f_c \text{ lb. per sq. in.}$$

t = thickness of arch (inches)
r = radius (inches)
f_c = residual compression (lb. per sq. in.)
T = crown-drop tension (lb. per ft. height of wall)
M_C = moment at C (lb.-in. per ft. height of wall)
M_E = moment at E (lb.-in. per ft. height of wall)





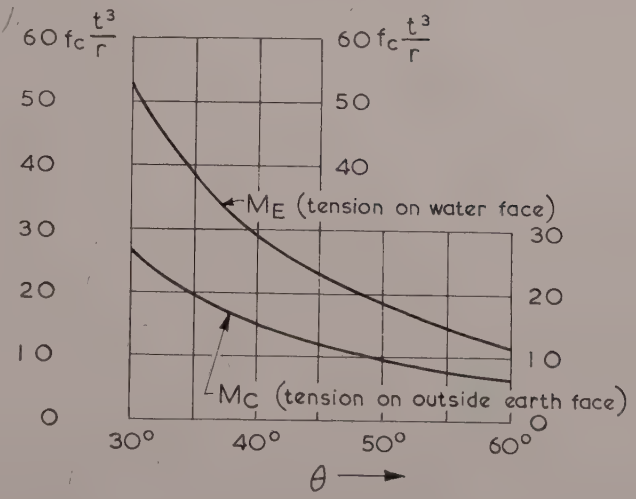


Fig. 14.4.

$$f_c = F_c - 0{\cdot}0243 f_c,$$

or $$f_c = 0{\cdot}975 F_c = 0{\cdot}975 \times 217 = 212 \text{ lb. per sq. in.}$$

Also from *Fig.* 14.4, $M_E = 22{\cdot}4 f_c \frac{t^3}{r}$ and $M_c = 11{\cdot}6 f_c \frac{t^3}{r}$.

$$M_E = 22{\cdot}4 \times 212 \times \frac{6^3}{120} = 8{,}550 \text{ lb.-in. per foot of height.}$$

$$\text{Stresses due to this moment} = \pm 8{,}550 \times \frac{6}{12 \times 6^2} = \pm 119 \text{ lb. per sq. in.}$$

$$= \pm 0{\cdot}56 f_c.$$

The stresses at Sections E and D are $(+212 \pm 119)$ or $+331$ to $+93$ lb. per sq. in., and there is no tension anywhere in the arch.

In arched bridges not only must allowance be made for this crown-drop effect but also for shrinkage and fall of temperature (that is, assuming that the arch contracts

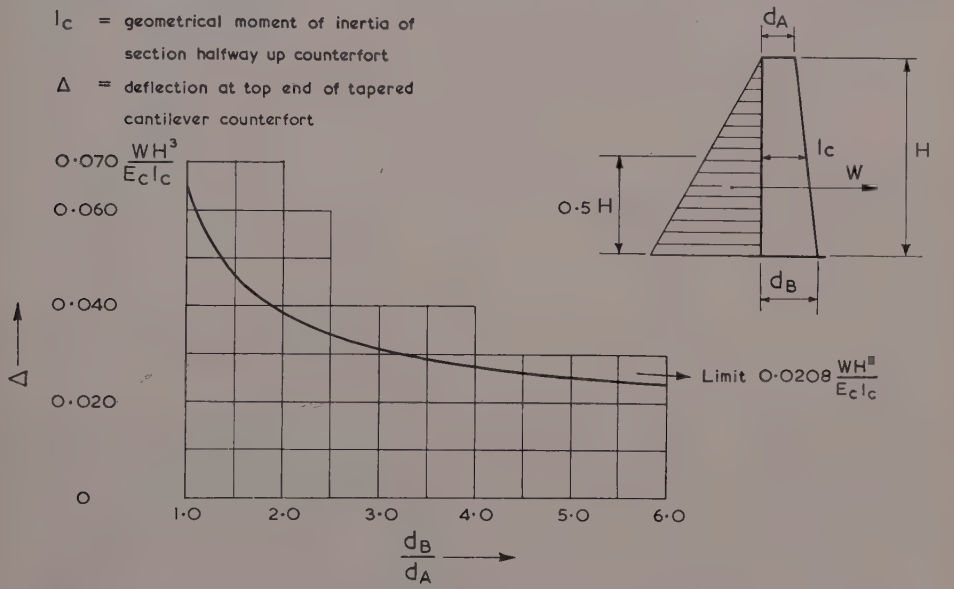


Fig. 14.5.

and the abutments do not move). In a reservoir the shrinkage should be negligible. The range of temperature should be small and will probably affect the whole wall. Even if these effects are neglected there is a calculated tension of 49 lb. per sq. in. in the foregoing example of a segmental arch, where θ is only 30 deg. and the writer would not recommend the use of such flat jack arches, even in a straight wall. In the second example where $\theta = 45$ deg. the calculated minimum compression is 93 lb. per sq. in.

Returning now to the curved wall shown in *Fig.* 14.1, any deflection or backward movement of the counterforts will increase the radius R and will increase the span ED pro-rata. If the counterforts sit on solid rock the only outward movement is their deflection. Many counterforts are tapered in elevation and in *Fig.* 14.5 the calculated deflection at the top end is given for different ratios of d_A to d_B. If the top ends of the counterforts in *Fig.* 14.1 move out 0·05 in. and the main radius R is 50 ft., the proportional increase is 1/12,000, equivalent to the shortening

due to a primary compression stress of 250 lb. per sq. in. in the small arches. This effect is a maximum where the water pressure is zero and is zero where the water pressure is a maximum. If, as before, $t = 6$ in., $r = 120$ in., and $\theta = 45$ deg., equivalent $F_c = 250$ lb. per sq. in.
Equivalent $f_c = 0{\cdot}975F_c = 244$ lb. per sq. in.

$$T = 116{\cdot}5f_c\frac{t^3}{r^2} \quad \text{and} \quad M_E = 22{\cdot}4f_c\frac{t^3}{r} \text{ (see } \textit{Fig.} \text{ 14.4).}$$

The stresses at E and D at the top of the wall are

$$-(0{\cdot}0243 \times 244) \pm (0{\cdot}56 \times 244) = -143 \text{ to } +131 \text{ lb. per sq. in.}$$

(tension on water face). If the same values of t and r are retained but θ is reduced to 30 deg., these stresses become -320 to $+264$ lb. per sq. in. These stresses are due solely to the outward movement of the top of the counterfort and reach this maximum value where the water pressure is zero.

In a square reservoir the outward deflection of counterforts next to those at the corners raises a problem (see *Fig.* 14.2). One of them moves say northward and the other moves west, while the corner counterfort moves north-west. The

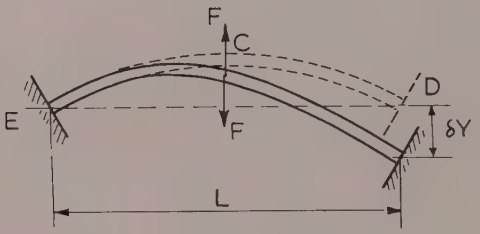


Fig. 14.6.

case of one counterfort moving away from the next and thus increasing the span has already been discussed. If one counterfort moves behind the line of the other by an amount δY, as shown in *Fig.* 14.6, this will cause neither moment nor thrust at the crown section C but will induce a shear force F which is approximately equal to $12E_cI\frac{\delta Y}{L^3}$. Writing I per foot $= t^3$, $F = 12E_c\delta Y\left(\frac{t}{L}\right)^3$. If $E_c = 3{,}000{,}000$ lb. per sq. in., $t = 6$ in., and $L = 144$ in., $F = 2{,}600\ \delta Y$ lb. per foot of height., and the moments M_E and M_D would be numerically equal to $2{,}600\delta Y \times 72$ lb.-in.

If $\delta Y = 0{\cdot}05$ in., then $M_E = 9{,}350$ lb.-in. This would cause stresses of $\pm\frac{9{,}350 \times 6}{12 \times 6^2} = \pm 130$ lb. per sq. in. at D and E. This result emphasizes the need to provide stiff counterforts with walls of this type and the need, in general, for strongly reinforcing the top part of the wall in addition to the bottom.

If a circular reservoir, as shown in *Fig.* 14.1, is on very soft ground and the floor is carried on long vertical piles, the feet of the counterforts are held in position by tensile stresses in the floor. Under these stresses the floor must stretch and the counterforts move outward. If the average tensile stress in the floor is F_c', the proportional increase in the span ED is F_c'/E_c.

If the primary compression in the arch due to applied water pressure is F_c, the stretching of the floor will increase the moments and tensions in the arches

in the ratio $\frac{F_c + F_c'}{F_c}$. If $F_c = 300$ lb. per sq. in. at the base of the wall and F_c' is 100 lb. per sq. in., with $t = 6$ in., $r = 120$ in. and $\theta = 45$ deg. (all as in the previous example),

$$f_c = 300 \times 0{\cdot}975 = 293 \text{ lb. per sq. in. and}$$
$$f_c' = 100 \times 0{\cdot}975 = 97{\cdot}5 \text{ lb. per sq. in.}$$

Stresses at points E and D are as follows.
At base of wall: $+293 \pm 0{\cdot}56(293 + 97{\cdot}5) = +510$ to $+74$ lb. per sq. in.
Half-way up wall: $+146 \pm 0{\cdot}56(146 + 97{\cdot}5) = +283$ to $+10$ lb. per sq. in.
At top of wall: $0 \pm 0{\cdot}56(0 + 97{\cdot}5) = \pm 55$ lb. per sq. in.
To these must be added the stresses caused by the outward deflection of the top end of the counterfort, which have already been calculated (assuming a deflection of 0·05 in.). Combining these conditions the total stresses at the top of the wall are as follows:
On water face at E, $-55 - 143 = -198$ lb. per sq. in.
On outside face at E, $+55 + 131 = +186$ lb. per sq. in.

From the values in *Fig.* 14.4 the maximum allowable deflection at the top end of a cantilevered counterfort in a circular reservoir of the type shown in *Fig.* 14.1 can be calculated.

If $R = 50$ ft. $= 600$ in., $r = 15$ ft. $= 180$ in., $t = 6$ in., and $\theta = 30$ deg., and the deflection of top end of counterfort is Δ in., from *Fig.* 14.4,

$$T = 593 f_c \frac{t^3}{r^2} = 593 f_c \times \frac{6^3}{180^2} = 3{\cdot}95 f_c.$$

Tensile stress due to $f_c = \frac{3{\cdot}95 f_c}{6 \times 12} = 0{\cdot}055 f_c.$

$$f_c = F_c - 0{\cdot}055 f_c. \quad \text{Thus } F_c = 1{\cdot}055 f_c.$$

Also from *Fig.* 14.4, $M_E = 52{\cdot}6 f_c \frac{t^3}{r} = 52{\cdot}6 f_c \times \frac{6^3}{180} = 63 f_c.$

Tensile stress due to $M_E = 63 f_c \times \frac{6}{12 \times 6^2} = 0{\cdot}876 f_c.$

Total tensile stress (on water face) $= 0{\cdot}055 f_c + 0{\cdot}876 f_c = 0{\cdot}931 f_c = 0{\cdot}885 F_c.$
Equivalent F_c corresponding to increase in radius (and increase in span ED)

$$= 3{,}000{,}000 \times \frac{\Delta}{R} = 3{,}000{,}000 \times \frac{\Delta}{600} = 5{,}000\Delta.$$

If the maximum calculated tension is limited to 300 lb. per sq. in., $0{\cdot}885 F_c = 300$, and as $F_c = 5{,}000\Delta$, the maximum allowable Δ is 0·068 in. If therefore the outward movement of the top of the counterfort exceeds 0·068 in., the calculated tension in the arch at the top of the wall will exceed 300 lb. per sq. in. on the water face at points D and E. All the effects so far discussed tend to create tension on the outside (earth) face at midspan. The position of the horizontal reinforcement necessary is shown diagrammatically in *Fig.* 14.7.

So far these arches have been treated as walling units spanning from counterfort to counterfort and the counterforts have been considered as rectangular beams, but the arches and counterforts must act together and the lower part of the arched wall should have substantial vertical steel well anchored into the reservoir floor.

When the tank is empty earth pressure on the outside face causes tension in the

small arches, but under these conditions the arch is not a ' water-containing ' structure.

Segmental arches may be constructed on the slope, as shown in *Fig.* 14.8. This is a small-scale adaptation of the Ambursen dam. If W is the total weight of the

Fig. 14.7.

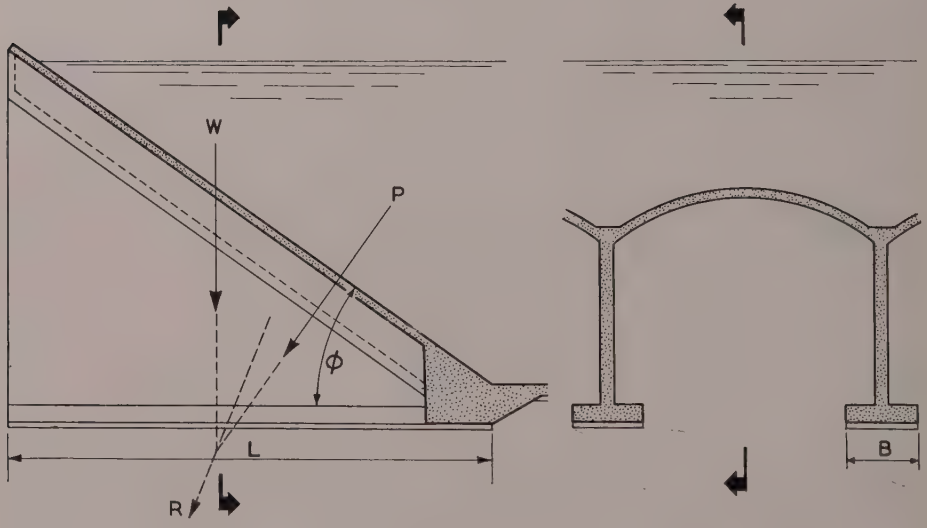


Fig. 14.8.

arch and wall and P is the total water pressure, the angle ϕ should be chosen so that the resultant R obtained by combining W and P passes close to the centre of area of the foundation. If the width B is constant the resultant should pass within the middle-third of L.

CHAPTER FIFTEEN

SPLAYED FLOOR AND WALL SLABS

In designs prepared prior to 1914, many continuous main beams were strengthened with splays or brackets at the internal columns. Occasionally, when the loading was uniform, the idea was extended to floor slabs or wall slabs such as in the case of retaining walls, square silos or tanks. These splays were about 0·167 times the span in length at each end, having the middle two-thirds of the span of uniform depth: the depth of the splays was such as to make the overall depth at the support 1·5 to 2·0 times the depth at midspan. The practice had largely died out by 1920 but the idea may well be re-introduced if the present insistence on low calculated tensile stresses in the concrete continues.

According to CP. 2007:1960, at the midspan section with compression on the water face, working stresses of 1,200 and 18,000 lb. per sq. in. are permitted with a safe moment of resistance of 250 bd_1^2, if the slab is 9 in. or more in thickness.

Using a 9-in. slab with 1·5-in. cover and $\frac{3}{4}$-in. diam. bars, $d_1 = 7{\cdot}125$ in.,

Safe $M = 250 \times 12 \times 7{\cdot}125^2 = 152{,}500$ lb.-in. per foot width.

If a slab of constant thickness is used the safe resistance moment with tension on

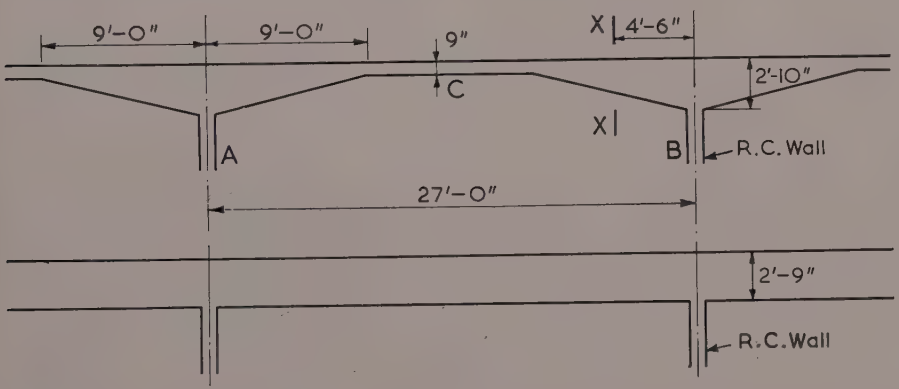


Fig. 15.1.

the water face is only $60bd^2$ or $60 \times 12 \times 9^2 = 58{,}300$ lb.-in. per foot width. Since the value of the moment at midspan is only $\frac{1}{24}wL^2$ while the moment at the support is $\frac{1}{12}wL^2$, the resistance available for the reversed moment is only 19 per cent. of what is required. If the thickness of the slab at the support is increased, the distribution of the bending moment is disturbed. With no splays the reversed moment is $0{\cdot}083wL^2$. With splays $0{\cdot}167L$ long and such that d_A (the overall depth at the supports) is twice d_C (the overall depth at midspan), the reversed moment is $0{\cdot}097wL^2$. With splays $0{\cdot}33L$ in length and with $d_A = 4d_C$, the reversed moment is $0{\cdot}114wL^2$. In *Figs.* 15.1 and 15.3 an extreme design is shown for a splayed section slab to carry a 10-ft. depth of water over a span of 27 ft. and the corresponding uniform section with the same loading and span.

In the splayed slab, $\frac{d_A}{d_C} = \frac{34}{9} = 3{\cdot}78$. Length of splay $= 9$ ft. $= 0{\cdot}33L$.

From tables, reversed $M = 0{\cdot}114wL^2$ (see *Fig.* 15.2). Positive moment at midspan $= 0{\cdot}011wL^2$. This is a very low figure and it is recommended that any

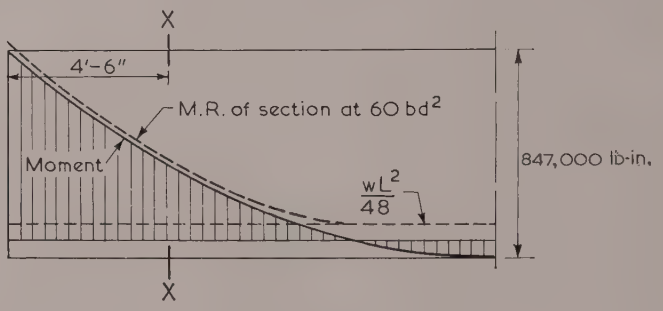


Fig. 15.2.

such case should be designed for not less than $\frac{1}{48}wL^2$ or $0{\cdot}0209wL^2$. The weight of the splays comes near the supports and scarcely affects the moments.

$$\frac{wL^2}{48} = (625 + 175) \times 27^2 \times \frac{12}{48} = 146{,}000 \text{ lb.-in.}$$

$$A_{st} = \frac{146{,}000}{18{,}000 \times 0{\cdot}83 \times 7{\cdot}125} = 1{\cdot}37 \text{ sq. in.,;}$$

say $\frac{3}{4}$-in. diam. bars at 4-in. centres.

Reversed $M = 0{\cdot}114 \times 800 \times 27^2 \times 12 = 796{,}000$ lb.-in.

Minimum d for resistance moment of $60bd^2 = \sqrt{\frac{796{,}000}{60 \times 12}} = 33{\cdot}3$ in.

$$A_{st} = \frac{796{,}000}{12{,}000 \times 0{\cdot}9 \times 32} = 2{\cdot}32 \text{ sq. in.}$$

With a slab of uniform thickness the reversed moment is $\frac{1}{12}wL^2$

$$= (625 + 413) \times 27^2 \times \tfrac{12}{12} = 756{,}000 \text{ lb.-in.}$$

Minimum d for resistance moment of $60bd^2 = \sqrt{\frac{756{,}000}{60 \times 12}} = 32{\cdot}4$ in.

$$A_{st} \text{ at midspan} = \frac{378{,}000}{18{,}000 \times 0{\cdot}9 \times 31} = 0{\cdot}75 \text{ sq. in.}$$

$$A_{st} \text{ at support} = \frac{756{,}000}{12{,}000 \times 0{\cdot}9 \times 31} = 2{\cdot}26 \text{ sq. in.;}$$

say 1-in. diam. bars at 4-in. centres.

Details of these two designs are shown in *Fig.* 15.3. The quantities of concrete and steel required per foot length for the splayed section are 39 cu. ft. and 346 lb., respectively. For the uniform section the equivalent values are 74 cu. ft. and 330 lb., respectively. The shuttering for the uniform section would be simpler but heavier and the extra dead weight (195 lb. per sq. ft.) might increase the cost of the foundations.

On figures then, the splayed section is much the cheaper but the reader is advised not to use such heavily-splayed sections until he has studied their behaviour as continuous beams and as members of structural frames. (All the necessary co-efficients can be found in the writer's ' Frame Analysis '.)

If the splay is restricted to double the depth at midspan and a minimum thickness of 9 in. is observed, the section shown in *Fig.* 15.4 is obtained. The reversed moment (from tables) is $0{\cdot}102wL^2$ and the moment of resistance, with a tensile stress of 300 lb. per sq. in., is about

$$60 \times 12 \times 18^2 = 234{,}000 \text{ lb.-in.}$$

or

$$0{\cdot}102wL^2 \times 12 = 234{,}000 \text{ lb.-in.}$$

This slab will therefore carry a 10-ft. depth of water over a span of 16 ft. or a 20-ft. depth of water over a span of 11 ft. 9 in.

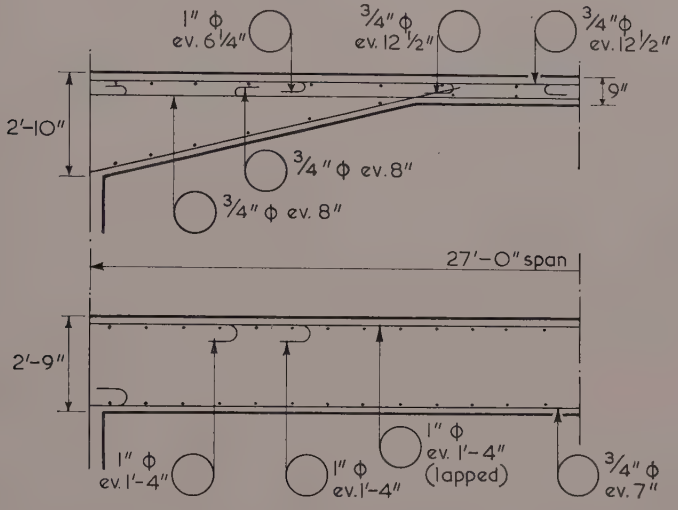


Fig. 15.3.

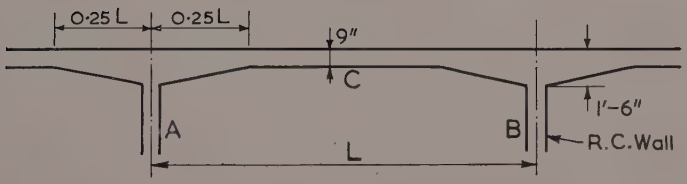


Fig. 15.4.

If the Code requirements are ignored and permissible stresses of 1,200 and 16,000 lb. per sq. in. are adopted for the reversed moment, a 16-in. slab will carry a 10-ft. depth of water over a span of 27 ft., against the 33-in. slab, shown in *Figs.* 15.1 and 15.3, needed to comply with CP. 2007: 1960.

In addition to vertical loading, most suspended floors have to carry some direct tension from the walls of the tank and the design must be strengthened as necessary to take the additional stresses.

When dealing with a uniform section it is only necessary to check the maximum value of the reversed moment. When a deep splay is provided, at least one intermediate section such as Section X–X in *Figs.* 15.1 and 15.2 must also be checked.

If, instead of increasing the strength of the support section the process is reversed and the bending strength is reduced to zero by introducing a construction joint with no top steel at this point (as in *Fig.* 15.5), the moment at midspan will be $\frac{1}{8}wL^2$. To carry a 10-ft. depth of water over a span of 27 ft. with stresses of 1,200

and 18,000 lb. per sq. in. will require a 20-in. slab reinforced with $1\frac{1}{8}$-in. diam. bars at $3\frac{1}{4}$-in. centres.

Generally speaking the writer is not greatly in favour of splayed sections but

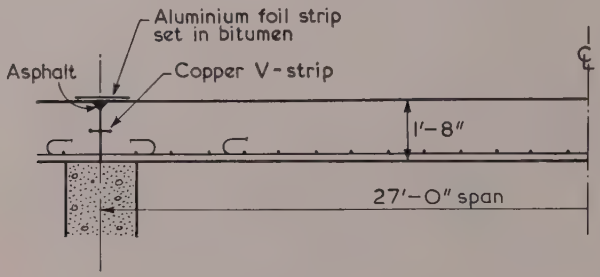


Fig. 15.5.

in actual practice so many different considerations may influence a decision and so many local difficulties may arise on site (particularly in under-developed countries) that occasions must occur when they offer the best solution.

CHAPTER SIXTEEN

RESERVOIR FLOORS

All the designs of floor discussed in the following have been built and have proved successful on many sites. Most, if not all of them, have given trouble on other sites. In a few cases the ground may have been softer, more variable or more easily compressible than anticipated, imposing bending moments and shearing forces on the floor slab, for which it was never designed. In many cases this is clearly not the explanation and the success (or failure) of a particular floor has led engineers to recommend (or condemn) the design to which this particular floor was built. From the general disagreement that has ensued it is clear that there must be other important factors that some engineers have failed to take into account in jumping to a conclusion and arguing from the particular to the general. These factors are discussed later when the main types of floor have been described.

It is much easier to clean out a tank or reservoir if the floor is laid to a fall. A slope of 1 inch in 10 feet is sufficient. In forming a sump or drainage trench no backfilling of earth must be allowed. Wherever soil has been removed in excess of the net volume occupied by the reinforced concrete walls, this excess must be filled back with mass concrete.

SUPPORTED FLOORS

Floors on Good Ground

Good ground includes very hard strata where no measurable settlement will develop and fairly hard strata where the settlement will be very small and (more importantly) uniform over the whole site. In either case the floor slab carries no effective structural loading and is free from structural bending moments. A 5-in. thickness of 1 : 1½ : 3 concrete, if not too dry and not too wet, is non-porous for a head of 10 ft. of water while a 6-in. slab is non-porous for a head of 20 ft. and a 7-in. slab for a head of 30 ft. These are conservative figures based on practice and are not taken from small-scale laboratory research. It is a simple matter therefore to ensure that the floor is non-porous.

Only one problem then remains—the problem of dealing with shrinkage and movement due to temperature change. Shrinkage may be reduced by careful curing during the early life of the concrete but cannot altogether be eliminated. Shrinkage is a three-dimensional problem. With a 6-in. or 8-in. floor slab, vertical shrinkage seems to produce no adverse effects. For ease in screeding, floors are usually laid in strips 15 ft. or 16 ft. wide and there is both longitudinal and transverse shrinkage. To add to the problem there is also the primary cooling shrinkage caused by the heat generated by the cement during the processes of setting and hardening. This may be a major problem in a really thick slab and is always a problem in a thick wall but is only a minor problem in a thin floor.

Under average air storage conditions a total shrinkage of 400×10^{-6} may be expected, or 0·48 in. in a length of 100 ft. after a period of 18 months. Only some 5 to 7 per cent. of the total shrinkage develops in the first week after placing and only about 25 per cent. in the first month.

Laying a floor in alternate bays or in chess-board squares with seven-day intervals between placing alternate bays will eliminate 50 per cent. of the primary cooling shrinkage but does not control 95 per cent. of the hardening shrinkage.

The figure of 400×10^{-6} will clearly apply to a road slab where the provision of contraction joints at 60-ft. centres is not unusual. The proportion of this value that may be eliminated in a reservoir floor depends on how well the curing is carried out during construction, the length of time the reservoir may stand empty, and what type of roof (if any) is provided. To estimate the magnitude of the effect assume that the shrinkage may amount to 200×10^{-6} or 0·24 in. in a length of 100 ft.

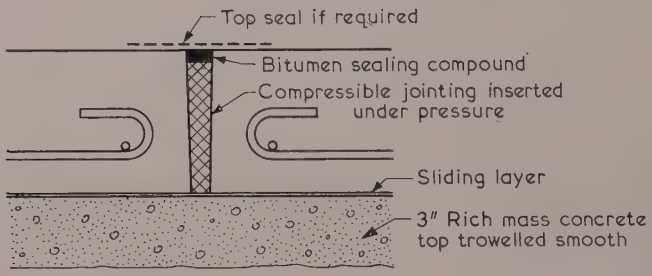

Top seal if required
Bitumen sealing compound
Compressible jointing inserted under pressure
Sliding layer
3" Rich mass concrete top trowelled smooth

Fig. 16.1.

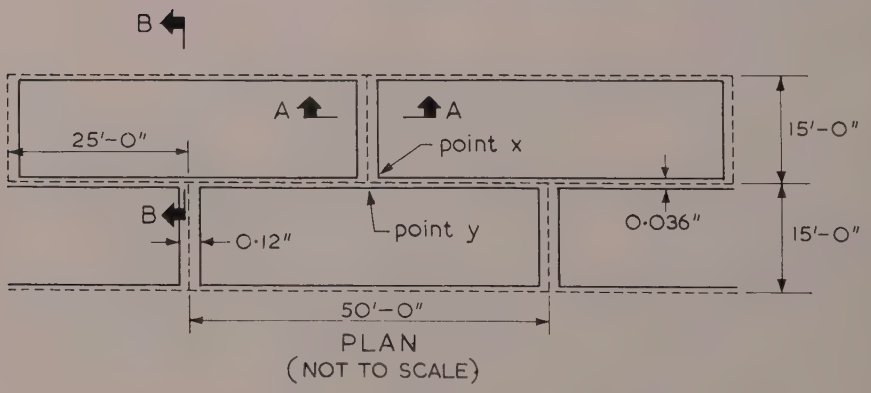


Fig. 16.2.

(assuming that the floor slab is supported on a frictionless base and is absolutely free to expand and contract in both directions).

There are two main schools of thought. The first of these is shown in *Figs.* 16.1 and 16.2. Each bay of floor is completely isolated from adjacent bays and is mounted on a sliding layer so that it can (theoretically) expand and contract freely in all directions. The sliding layer might formerly have been tarred paper but would now probably be a sheet of some synthetic polymer. (Some polyester films wrinkle when laid.)

If the bays are 50 ft. by 15 ft. as shown in *Fig.* 16.2, the joints will open 0·12 in. and 0·036 in. (assuming that the centre-point of each bay remains in its original position). Note that point *x* moves a distance 0·036 in. away from point *y* in a north-south direction and a distance 0·06 in. in an east-west direction, and the jointing material must be able to take up both of these movements. The jointing

material might have been fabricated from paper and bitumen but may now be foamed plastic. As the material is required to expand it must be placed in the joint in a compressed state. However good the jointing material may be and whatever claims the makers may put forward, some additional safeguard (such as covering the joint with aluminium foil set in thick bitumen paint) is usually wise. If the floor slab is 6 in. in thickness and weighs 75 lb. per sq. ft., and is laid in bays 50 ft. long, the maximum tension per foot width occurs on Section B–B in *Fig.* 16.2. and is $25 \times 75 \times \mu$, where μ is the coefficient of friction of the sliding layer. If μ is 0·2, then the tension is 375 lb. or 52 lb. per sq. in.

Since the steel bars forming the reinforcement grow shorter they are in compression. With a strain of 200×10^{-6}, the stress induced is 6,000 lb. per sq. in. This compression tends to resist the movement due to shrinkage. If an accidental construction joint was provided down the centre of the panel on Section B–B in *Fig.* 16.2, the steel would have to carry the tension of 375 lb. At 12,000 lb. per sq. in., an area of 0·031 sq. in. would be required. Under good conditions a floor

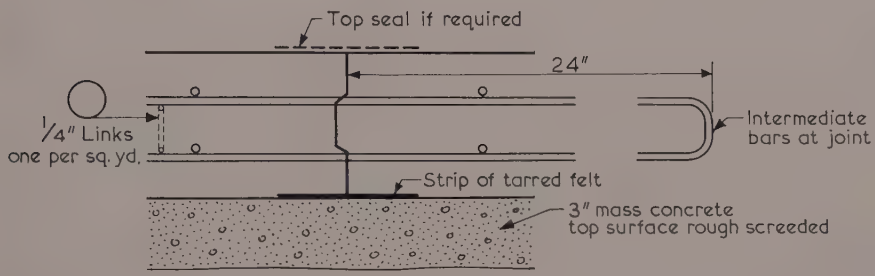


Fig. 16.3.

of this type may be reinforced with $\frac{3}{8}$-in. diam. bars at 12-in. centres in both directions. This is only 0·15 per cent.

The second main school of thought is illustrated in *Fig.* 16.3. This school of thought, instead of making provision for free movement, as in *Fig.* 16.1, seeks to prevent general movement and, if cracks occur, to spread them as numerous very fine cracks too small to start corrosion or cause leaks. Assuming that the floor slab is anchored down to the layer of mass concrete and that this is anchored to the ground the strain in the slab is 200×10^{-6}. If the elastic modulus in tension is 2,000,000 lb. per sq. in. then the stress is $200 \times 10^{-6} \times 2{,}000{,}000 = 400$ lb. per sq. in. Since the steel bars are neither longer nor shorter than they were originally there is no stress in them except at the construction joint. If the slab is 6 in. in thickness, the total tension per foot width is $6 \times 12 \times 400 = 28{,}800$ lb. If the steel may be stressed to 13,500 lb. per sq. in.,

$$A_{st} = \frac{28{,}800}{13{,}500} = 2{\cdot}14 \text{ sq. in.}$$

while 0·3 per cent. of the area of the section is only 0·216 sq. in. If the floor is normally reinforced with $\frac{3}{8}$-in. diam. bars at 12-in. centres top and bottom this gives 0·22 sq. in. Even if intermediate bars are added at the construction joints, the area is only 0·44 sq. in.

In a covered reservoir built on reasonably good ground it is easily possible to complete the walls and roof before the floor slab is laid. (The discharge from a concrete pump will pass through a small manhole.) The floor is then not only protected from wind, frost and sun during and after construction, but it can be flooded with water as soon as it is complete. If this reduces the effective shrinkage to 100×10^{-6}, the stress in the concrete falls to 200 lb. per sq. in. If an area of 0·44 sq. in of steel crosses the construction joint, the calculated stress in the steel would be 33,000 lb. per sq. in. Provided the floor were laid in bays not exceeding 15 ft. by 30 ft. it would be sound under these conditions.

It is clear from the general disagreement that exists and from the fact that there is no generally accepted standard practice, that the calculations prepared for the two types of floor give an incorrect or incomplete picture of what actually happens, and that there are other factors that must play an important part in the design and construction of reservoir floors.

As an example of differing points of view in CP.2007:1960 a collection of no less than 17 different details are shown which may be used in making joints in a floor slab. From elementary logic it must follow that the enthusiastic exponent of any one of these types of joint must consider the other sixteen types either unnecessary or inefficient, and the collection in CP.2007 is by no means exhaustive. The technical description given to a particular joint cannot in any way influence the way it actually functions. Whether it is described as a ' construction joint ', ' contraction joint ', ' expansion joint ', etc., will make not the slightest difference to how it behaves in practice.

Possible factors not so far discussed include the concrete mix, the type of cement used, and workmanship. It has been found in practice that the effects of shrinkage are reduced to a minimum when a nominal $1 : 1\frac{1}{2} : 3$ mix is used; this has been discussed in Chapter 1. The type of cement used probably plays an important part. More site research is needed on this point. Any cement that becomes hard and brittle at an early age should be avoided. If it were possible to find a slow-hardening cement that would make a $1 : 1\frac{1}{2} : 3$ concrete with a plastic yield in tension of more than 200×10^{-6} under a tension of less than 200 lb. per sq. in. the problem of shrinkage would not arise. Examination of structures built well before 1914 suggests that they were built with cement of this type or cement having properties which produced the same effect.

The major influence that workmanship can play was clearly demonstrated under wartime conditions when a number of structures were built by different contractors from identical drawings and specification. It was further demonstrated when similar buildings stood up (or failed to stand up) to bombing depending on the excellence (or otherwise) of their workmanship.

Designing a sound floor consists not merely in choosing the right kind of joint and the right amount of reinforcement, but includes ensuring that all other conditions are as favourable as possible. On good ground, two-layer and three-layer floors should not be necessary if the specification is sound and continuous technical supervision is available.

Gunite has been used, particularly in the United States, for the construction of reservoir floors. It produces a dense slab. From 3 in. to 4 in. is a reasonable thickness. Mesh reinforcement is recommended for this class of floor. Good practice in this method of forming the floor is exemplified by a recent case in which $1\frac{1}{2}$ in. of gunite was put down first; on this the mesh was laid and a 2-in. coating shot on.

Floors on Patchy Ground

Some types of glacial deposit are very patchy and may vary from sand to clay to shingle and back to sand within a length of a few feet. Although the maximum settlement may be very small, it may not be uniform and bending moments will then develop in the floor slab. The additional reinforcement required depends on the relative compressibility of the various types of soil, a subject about which little is known despite the fact that it is a major factor in road design. In the present state of ignorance the designer can only guess what amount of steel is required and must decide, from a careful examination of the soil, whether this should

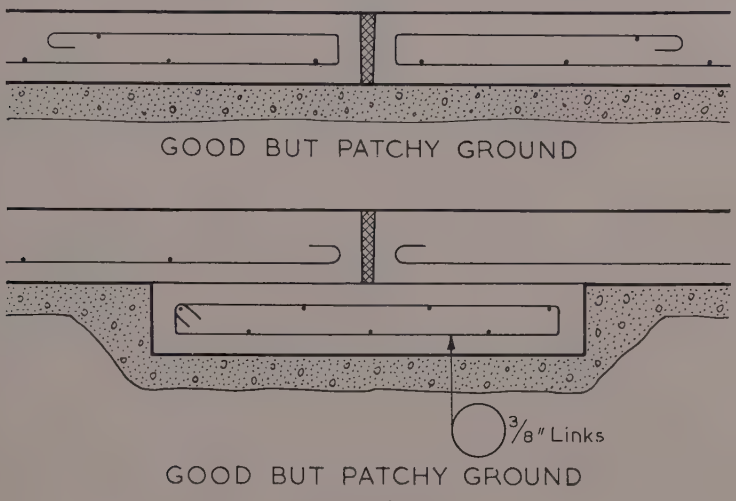


Fig. 16.4.

all be in one layer near the bottom of the slab or split into a top layer and a bottom layer.

If a softer patch occurs under a joint the slab must cantilever at this point.

If the variation is neither severe nor sudden, the detail in the upper part of *Fig.* 16.4 may be used. Under less favourable conditions the bolster slab (shown in the lower part of the figure) may be added. The provision of dowel bars half set in cardboard tubes, at present so popular in road design, is a detail that the author would not recommend.

A 6-in. slab with 0·3 per cent. of reinforcement in both directions is the amount of average reinforcement that ground of this type calls for. This may be placed in a single bottom layer or split into two layers of 0·15 per cent. near the top and bottom faces of the slab.

Foundations on Compressible Ground

Where the ground is good and uniform but compressible (either elastically or in terms of consolidation settlement), consideration must be given to the relative vertical movement between the floor slab, the wall base and the column foundations (if any). The problem will be most severe where there is a roof heavily loaded with earth. A depth of 20 ft. of water (including the floor) gives a load of about 0·6 tons per sq. ft. If the bases under the columns are designed for 2 tons per

sq. ft., this is more than 3 times the loading under the floor slab. Under these conditions a joint of the type shown in *Fig.* 16.5 is usually made between the floor slab and the base slabs of the walls and columns.

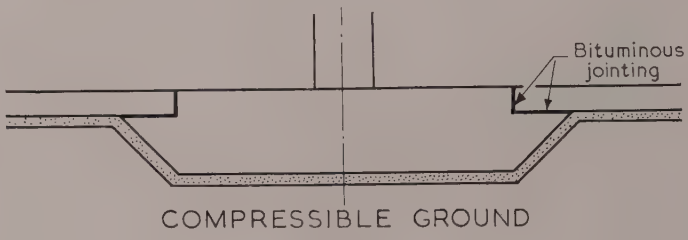


Fig. 16.5.

Two-layer and Three-layer Floors

In road construction the slab may be laid in two layers, but the placing of the upper layer follows the lower almost immediately, forming in effect a single-layer slab. Reservoir floors have been laid in layers with a definite time interval between, thus forming two (or even three) separate slabs. The layers may be separated by bituminous paint or by a layer of bituminous fabric, glass-fibre scrim set in bitumen, synthetic polymer sheet, etc. The main idea of a double-layer floor is that the upper layer covers the joints in the lower layer.

The floor construction shown in *Fig.* 16.6 is intended for a covered reservoir.

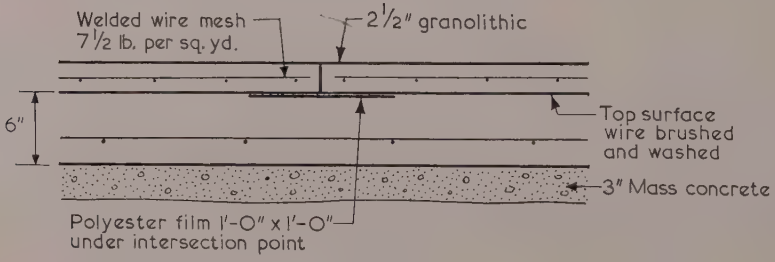

Welded wire mesh 7½ lb. per sq. yd.
2½" granolithic
6"
Top surface wire brushed and washed
3" Mass concrete
Polyester film 1'-0" x 1'-0" under intersection point

Fig. 16.6.

The mass concrete and 6-in. floor slab are put down before the walls and roof are built. The 2½-in. layer of granolithic is laid after the roof is complete and fully loaded. Both the 5-in. slab and the 2½-in. finish are laid in panels not exceeding 15 ft. by 30 ft. and the construction joints in both are set out on the drawings so that they break joint. Joints in the mass concrete layer should also be out of line with joints in the upper layers. The most vulnerable points are where the joints in the two layers cross. These points could be sealed with small sheets of synthetic polymer (say a foot square) glued to the lower layer with synthetic resin. The joints in the upper layer could be sealed with long cover strips but this should not be necessary.

Instead of the arrangement shown in *Fig.* 16.6, two separate 4-in. slabs could be used.

In the construction of the floor of the Admiralty oil fuel reservoirs at Rosyth a slab comprising 2 ft. of plain concrete with a surfacing of fine concrete was used. On testing the floor with water there was an increase in the flow through the sub-

drains, and to prevent any possibility of oil escaping when the tanks were filled the whole floor was covered with 6 in. of reinforced concrete in the proportions of 1 : 1½ : 2½. This slab was laid in squares, alternate squares in alternate rows of squares being laid first. After these had hardened the intermediate squares were filled in, forming alternate strips of concrete across the floor with intermediate uncompleted strips. The squares in the latter strips were then concreted on the alternate bay system, beginning with those adjacent to the first squares laid. Alongside the walls a strip of floor was omitted until the remainder of the floor had been concreted, and the reinforcement of this strip was bonded into a chase cut in the face of the walls. The result was entirely satisfactory.

In recent years it has become more general to lay the floor in two courses. In one good example the lower course was 5 in. in thickness and contained a mesh of ½-in. diam. bars at 12-in. centres. Above this a light canvas with lap joints was laid and painted over with asphalt. The upper course, 3 in. of concrete reinforced with ¼-in. diam. bars at 5-in. centres, was laid on the canvas. Between the 3-in. layer of concrete and the wall a 2-in. asphalt joint was made.

Water-bars in Floors

The placing of water-bars in wall joints is comparatively simple but in floor slabs it is difficult. In *Fig.* 16.7 a flexible synthetic water-bar is shown, both as imagined by the designer and as it is most likely to be placed in practice. A stout galvanished

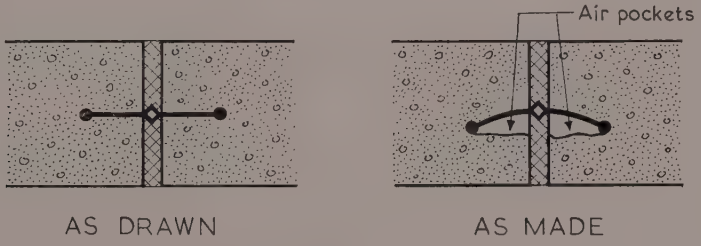


Fig. 16.7.

steel or copper jointing strip would be less likely to get bent but the use of either would require a workman with a special tool to be specially detailed to scrape the air pockets out from under the wings of the jointing strip.

Where movement is expected a bent copper strip of the type shown in *Fig.* 16.8 may be used.

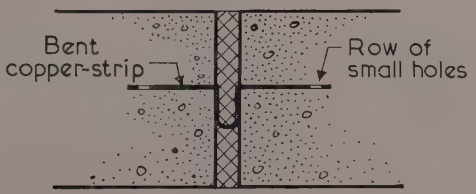


Fig. 16.8.

With all types of jointing strips the arrangement of joints in *Fig.* 16.2 cannot be used, owing to the relative movement of points *x* and *y*, and the joints must line up in both directions. Even then there is difficulty at the intersection of the joints.

This may be overcome by using a special junction piece that can be pulled open in both directions.

Waterproof Floor Linings

A $\frac{1}{2}$-in. rendering of fine sand and cement with some waterproofing additive is simple and cheap to lay, and looks clean and neat. It may remain in place as long as the reservoir is full, but after the reservoir has stood empty for a few weeks it will crack and spall off. In any case, since it is non-plastic it will split immediately a crack develops in the slab below it and it can therefore serve no useful purpose.

Asphalt, unless it is plastic and can be guaranteed to remain plastic, is also useless on a well-made concrete floor, for brittle asphalt will split as soon as a crack develops or a construction joint opens. It is claimed that certain types of bitumen will remain plastic indefinitely, and a layer of such bitumen reinforced with glass-fibre scrim may be used provided that it does not affect, and is not affected by, the contents of the tank.

A layer of gunite reinforced with welded steel mesh is generally very sound and very dense. Since it is expensive it is normally only used in this country when major repairs are required on a badly designed or badly built tank.

Sumps

Most floors are laid to falls. Large tanks and reservoirs are usually provided with a wash-out pipe which connects to a bellmouth or hopper-head set in the floor and provided with a grating. Sometimes a sump is provided. The most important thing to remember in designing a sump is that it is, of course, placed

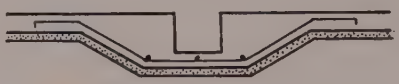


Fig. 16.9.

at the lowest point of the floor and that any rain that runs off the surface of the excavation collects at this point and may bring a quantity of mud with it. Before the excavation for the sump is commenced the reinforcement must be bent and assembled into a cage and the shuttering also must be made and assembled into a unit so that the sump can be built quickly.

A small sump, say 1 ft. 6 in. by 1 ft. 6 in. by 1 ft. 3 in. deep, may be made as

Fig. 16.10.

Fig. 16.9, the hole being lined with 3 in. of mass concrete immediately it has been excavated.

A larger sump, say 3 ft. by 3 ft. by 3 ft. deep, may be made as *Fig.* 16.10. If the ground is very hard the sides of the excavation may be trimmed vertical and the reinforced concrete walls may be cast against the face of the excavation (allowing 3 in. of concrete cover outside of the outside bars). Alternatively, the sides of the

excavation may be close timbered and the walls cast against the timbering which is left in. Both of these methods will produce the result shown in the left-hand side of the figure. On the right-hand side of *Fig.* 16.10 the sides of the excavation are shown trimmed to a natural slope. The bottom of the excavation is then covered with 3 in. of mass concrete. On this is stood a box of rough shuttering as big as the outside of the sump and mass concrete is filled in outside this box. This mass concrete then acts as shuttering for the outside face of the reinforced concrete walls. Whatever method is adopted, no back filling must be allowed outside the walls of the sump. The same restriction applies to any pipes running below the

Fig. 16.11.

floor. The trenches in which these pipes are laid must be filled back with mass concrete.

As far as the calculated strength of the floor and walls is concerned, a thickness of 6 in. is ample for a small sump of this type and this thickness may be used in a dry climate. In an unpredictable climate where a sudden storm may fill the hole with mud and water a 9-in. thickness with a minimum of 2 in. of concrete cover makes steel fixing and concreting easier, quicker and sounder. Where a '9-in.' wall is cast against the earth face or against timbering, it will be $10\frac{1}{2}$ in. in thickness with $3\frac{1}{2}$ in. of concrete cover on the outside face.

Both the vertical and horizontal steel in the sump in *Fig.* 16.10 should consist of bars bent as shown in *Fig.* 16.11. For a 3-ft. by 3-ft. by 3-ft. sump, $\frac{3}{8}$-in. diam. bars at 8-in. centres in both directions near both faces are ample for depths up to 30 ft.

SUSPENDED FLOORS

Reservoir Floors on Poor Ground

Most reservoirs or large tanks contain water and are between 10 ft. and 40 ft. deep. With a depth of 40 ft. the total loading on the ground is about 1·2 tons per sq. ft. Any reasonably sound ground will safely carry this. A nominal slab say 6 in. in thickness laid on 3 in. of concrete blinding will suffice.

If the top 2 ft. or 3 ft. of soil is soft or if the greater part of the area is sound with isolated soft patches, the poor ground may be removed and replaced with weak concrete. If there is no suitable foundation within 6 ft. of the top of the floor, the cost of removing all the soft ground and replacing with weak concrete becomes excessive and it is cheaper to go down to firm ground with piers, walls or piles and span the floor between them.

Floors Spanning One Way between Sleeper Walls

If there is a sound bearing stratum at less than about 10 ft. down, trenches may be excavated and filled with mass concrete to form sleeper walls on which the slab is carried. If the tank is 20 ft. deep, the total load including the floor

will be about 0·65 tons per sq. ft. If the foundation layer will safely carry a net loading intensity of 3 tons per sq. ft. then the area of supporting wall must be about $\frac{1}{3} \times 0{\cdot}65$ or 22 per cent. of the floor area. If mass concrete walls 3 ft. in thickness are provided then they may be spaced at about 13-ft. 6-in. centres. With walls 2 ft. in thickness they may be spaced at about 9-ft. 6-in. centres. The excavation and mass concrete will be approximately the same in both cases but the 2-ft. walls will need about 50 per cent. more timbering and the excavation may cost more per cu. yd. Against this, the use of 2-ft. walls will allow a thinner and cheaper floor slab to be provided. If the sleeper walls are more than 6 ft. in height and the ground above the foundation stratum is really soft, they may look a little unstable. The reader must study site conditions and decide in each case whether 2-ft. or 2-ft. 6-in. mass concrete walls are or are not a sound proposition. In the following, walls 3 ft. in width will be assumed and calculations will be commenced by designing a slab of uniform thickness.

First the effective span of the slab must be determined. For an internal span (assuming that the wall does not rock on its foundation) the distribution of pressure between the underside of the slab and the top of the wall will be somewhat as shown in *Fig.* 16.12, depending on the relation between the stiffness of the slab and the

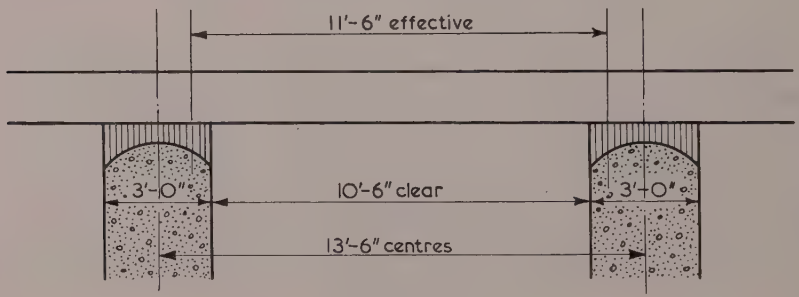


Fig. 16.12.

compressibility of the mass concrete. The effective span will be little more than the clear span of 10 ft. 6 in. and the design may be based on a span of 11 ft. 6 in. Slab spanning 11 ft. 6 in. to carry a depth of 20 ft. of water:

Imposed load: $20 \times 62{\cdot}5 = 1{,}250$ lb. per sq. ft.
Self-weight: say 200 ,, ,, ,, ,,
1,450 lb. per sq. ft.

Reversed $M = 1{,}450 \times 11{\cdot}5^2 \times \frac{12}{12} = 192{,}000$ lb.-in.

Minimum d for resistance-moment value of $60bd^2 = \sqrt{\dfrac{192{,}000}{60 \times 12}} = 16{\cdot}32$ in.;

say a 17-in. slab.

$$A_{st}, \text{ for stress of 12,000 lb. per. sq. in.} = \frac{192{,}000}{12{,}000 \times 0{\cdot}88 \times 15} = 1{\cdot}215 \text{ sq. in.};$$

say $\frac{5}{8}$-in. diam. bars at 3-in. centres.

$$A_{st} \text{ at midspan} = \frac{96{,}000}{18{,}000 \times 0{\cdot}9 \times 15} = 0{\cdot}396 \text{ sq. in.};$$

say $\frac{5}{8}$-in. diam. bars at 9-in. centres.

This is actually less than the minimum area of 0·3 per cent. specified by CP.2007: 1960, which requires $\frac{5}{8}$-in. diam. bars at 6-in. centres. This solution is shown in *Fig.* 16.13.

Many existing tanks have been designed to stresses of 1,000 and 16,000 lb. per sq. in., ignoring the calculated tensile stress in the concrete. Working to these stresses:

$$\text{Minimum } d_1 = \sqrt{\frac{192{,}000}{203 \times 12}} = 8{\cdot}88 \text{ in.; say } 11 \text{ in. overall.}$$

$$A_{st} \text{ over support} = \frac{192{,}000}{16{,}000 \times 0{\cdot}84 \times 9} = 1{\cdot}59 \text{ sq. in.}$$

$$A_{st} \text{ at midspan} = 0{\cdot}795 \text{ sq. in.}$$

This solution is shown in *Fig.* 16.14.

A compromise between *Figs.* 16.13 and 16.14 would make a sound design.

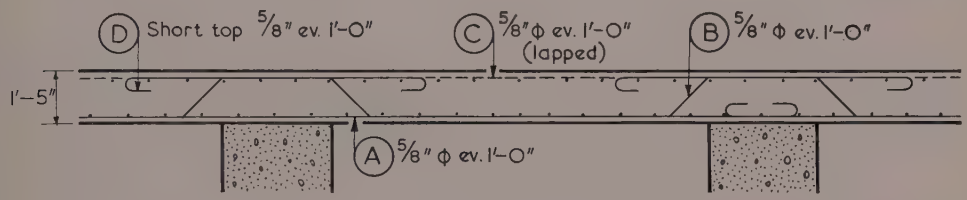


Fig. 16.13.

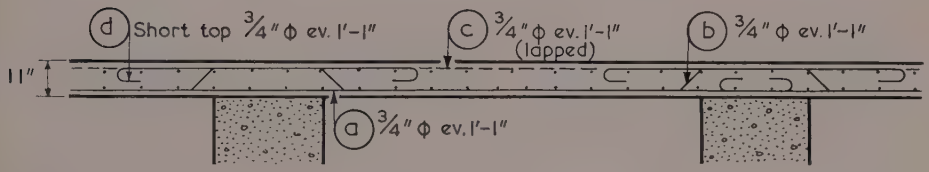


Fig. 16.14.

If concrete must be saved but the stresses specified by CP.2007:1960 must be observed a splayed section such as that shown in *Fig.* 16.15 may be used.

Imposed load: $20 \times 62{\cdot}5 = 1{,}250$ lb. per sq. ft.
Effective self-weight: say 130 ,, ,, ,, ,,
1,380 lb. per sq. ft.

Reversed $M = 1{,}380 \times 11{\cdot}5^2 \times 0{\cdot}102 = 18{,}600$ lb.-ft.

$$\text{Minimum } d \text{ for resistance moment of } 60bd^2 = \sqrt{\frac{18{,}600 \times 12}{60 \times 12}} = 17{\cdot}6 \text{ in.;}$$

say 18 in. overall.
Positive $M = 0{\cdot}023 \times 1{,}380 \times 11{\cdot}5^2 = 4{,}200$ lb.-ft.

$$A_{st} \text{ over support} = \frac{18{,}600 \times 12}{12{,}000 \times 0{\cdot}88 \times 16} = 1{\cdot}32 \text{ sq. in.}$$

$$A_{st} \text{ at midspan} = \frac{4{,}200 \times 12}{18{,}000 \times 0{\cdot}88 \times 7} = 0{\cdot}454 \text{ sq. in.}$$

The writer would not use a stress exceeding 16,000 lb. per sq. in. in the steel for a slab of this type, for which stress 0·5 sq. in. of reinforcement is needed.

With wide supports of this type two moment diagrams should be drawn --one for the assumed effective span (in this case 11 ft. 6 in.) and one for the clear span (10 ft. 6 in.) to make sure the top steel is extended sufficiently far out towards mid-span. These diagrams are drawn in *Fig.* 16.16.

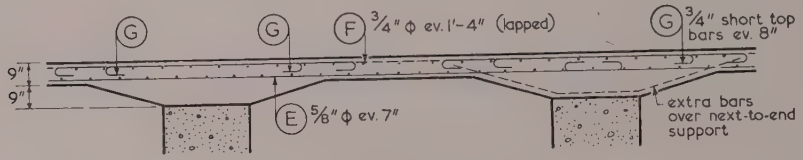


Fig. 16.15.

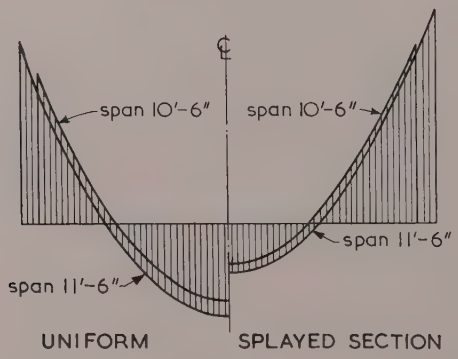


Fig. 16.16.

Fig. 16.17.

Most suspended floors have to carry some direct tension as well as bending moment and the floor will then require extra steel and possibly additional thickness. Construction joints may be made over the supports.

The reader is left to work out the alternative arrangement for 2-ft. sleeper walls spaced at 9-ft. 6-in. centres.

Instead of straight splays a curved soffit may be used (*Fig.* 16.17).

Floors between Sleeper Walls Spanning in Two Directions

If the top ground is very soft and the foundation stratum more than 10 ft. below the underside of the floor, mass concrete walls 3 ft. in thickness may look unstable. Instead of increasing the width, walls can be provided in both directions so that they support one another laterally (see *Fig.* 16.18). For a tank 20 ft. deep weighing 0·65 tons per sq. ft. on a foundation stratum capable of supporting a net intensity of loading of 3 tons per sq. ft., the sleeper walls being 3 ft. in thickness and spaced at L ft. by L ft., then

$$L^2 - (L - 3)^2 = \tfrac{1}{3} \times 0{\cdot}65L^2 \text{ or } L = \text{say } 26 \text{ ft.}$$

The floor is a mesh-panel slab spanning 26 ft. by 26 ft. Comparing this with a mesh-panel floor in a building, in the present case the corners are not held down structurally (except possibly by columns supporting the roof) but there is no question

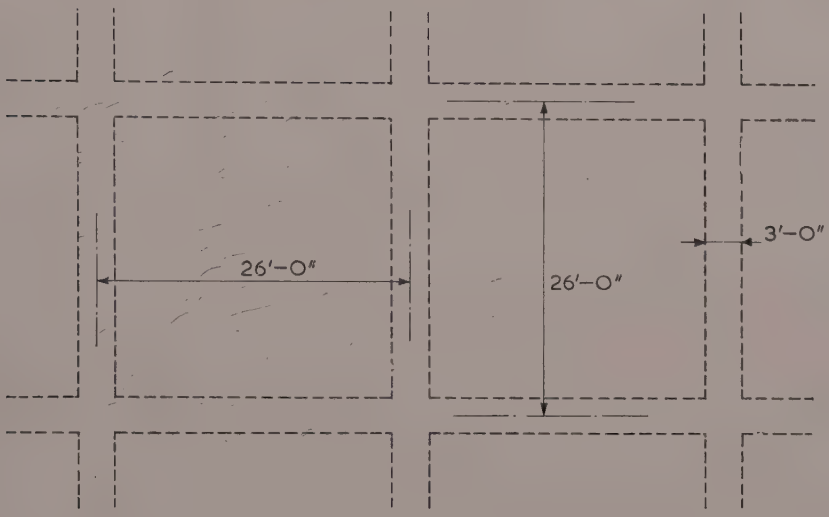


Fig. 16.18.

of unequal loading being applied. The ' beams ' supporting the edges of the panel are 3 ft. wide and they will not deflect at ' midspan '.

Assuming moments of $\pm\frac{1}{36}W$ for such a case,

$$W = 26 \times 26 \times 0{\cdot}65 \times 2{,}240 = 985{,}000 \text{ lb.}$$

$$M = \pm\frac{985{,}000 \times 12}{36} = \pm 328{,}000 \text{ lb.-in. per foot.}$$

Overall depth for reversed moment with resistance moment of $60bd^2$

$$= \sqrt{\frac{328{,}000}{60 \times 12}} = 21{\cdot}3, \text{ in.; say 22 in. (see } \textit{Fig.} \text{ 16.19).}$$

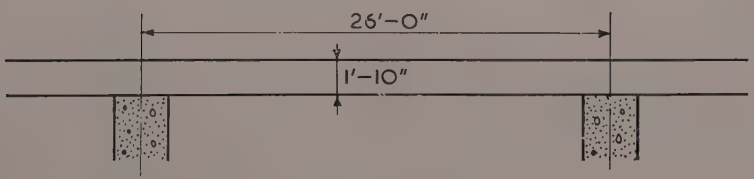


Fig. 16.19.

This compares with the 17-in. floor in *Fig.* 16.13, and there is no point in adopting a two-way span unless site conditions make it advisable. (If it were possible to make the thickness of the sleeper walls only 2 ft. the panel may be reduced in size to 18 ft. by 18 ft. and a 15-in. slab would be sufficient.)

In order to save a little concrete the ground can be trimmed as shown in *Fig.* 16.20, making the centre of the panel half as thick as the edges and design moments of $-\frac{1}{24}W$ and $+\frac{1}{48}W$, or −493,000 and +246,000 lb.-in. per foot may be adopted.

Overall depth required for reversed moment with resistance moment of $60bd^2$

$$= \sqrt{\frac{493{,}000}{60 \times 12}} = \text{say } 27 \text{ in.}$$

d_1 for positive moment $= \sqrt{\dfrac{246{,}000}{250 \times 12}} = 9{\cdot}06$ in., against 11·0 in. provided. Some details of these two designs are given in *Figs.* 16.21 and 16.22.

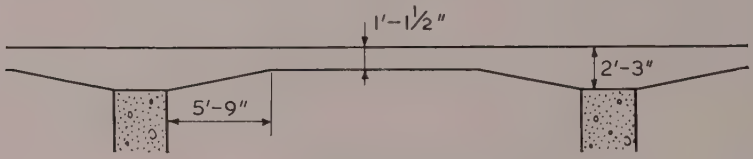


Fig. 16.20.

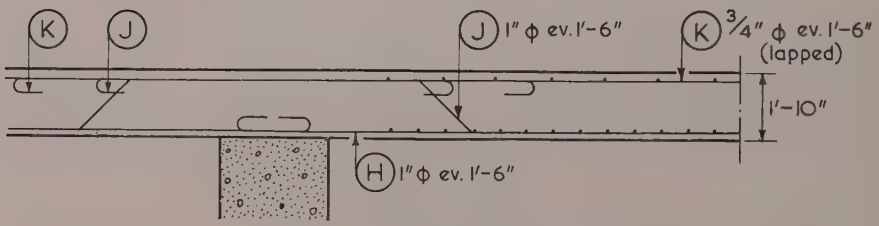


Fig. 16.21.

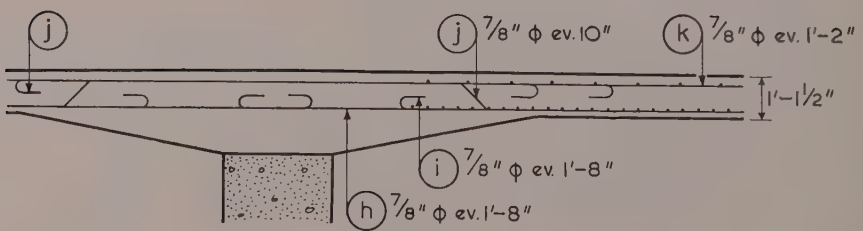


Fig. 16.22.

Floors Supported on Mass Concrete Piers

An expert miner can excavate a pier hole 4 ft. by 4 ft. in plan, but for average unskilled labour a hole about 5 ft. 6 in. by 5 ft. 6 in. is the minimum size advisable if the pier is more than 4 ft. deep. If the foundation stratum has an allowable bearing pressure of 8 tons per sq. ft., a pier 5 ft. 6 in. by 5 ft. 6 in. will carry 242 tons. If the piers carry the floor of a tank 30 ft. deep weighing 2,175 lb. or 0·97 tons per sq. ft., each pier will carry 250 sq. ft. of floor or a spacing of, say, 15 ft. 9 in. by 15 ft. 9 in.

If a sunk-panel flat-slab design is adopted the width of the beams is

$$0{\cdot}33 \times 15 \text{ ft. } 9 \text{ in.} = 5 \text{ ft. } 3 \text{ in.}$$

These would naturally be made as wide as the pier or 5 ft. 6 in. (66 in.) in width (see *Fig.* 16.23).

Total load on panel $W = 2{,}175 \times 15{,}75^2 = 540{,}000$ lb.

Reversed moment in Strip A $= -\dfrac{WL}{28} = -540{,}000 \times \dfrac{15{\cdot}75}{28} = -304{,}000$ lb.-ft.

For a resistance moment of $60bd^2$, minimum $d = \sqrt{\dfrac{304{,}000 \times 12}{60 \times 66}} = 30{\cdot}3$ in.; say 31 in. overall.

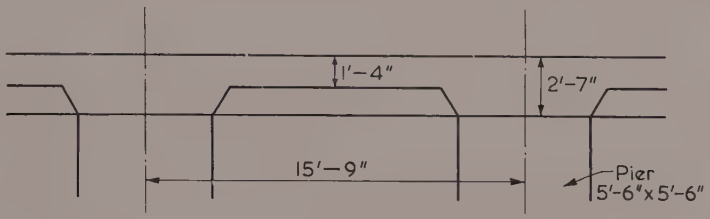


Fig. 16.23.

Shear strength around perimeter of pier at 100 lb. per sq. in.
$= 4 \times 66 \times (0{\cdot}88 \times 28{\cdot}5) \times 100 = 662{,}000$ lb. which exceeds W.

A_{st} for reversed M in Strip A, at 12,000 lb. per sq. in.

$$= \frac{304{,}000 \times 12}{12{,}000 \times 0{\cdot}88 \times 28{\cdot}5} = 12{\cdot}1 \text{ sq. in.; say sixteen 1-in. diam. bars.}$$

Positive moment in Strip A $= \dfrac{WL}{40} = 212{,}500$ lb.-ft.

A_{st} required, at 18,000 lb. per sq. in. $= 5{\cdot}56$ sq. in.; say thirteen $\frac{3}{4}$-in. diam. bars.

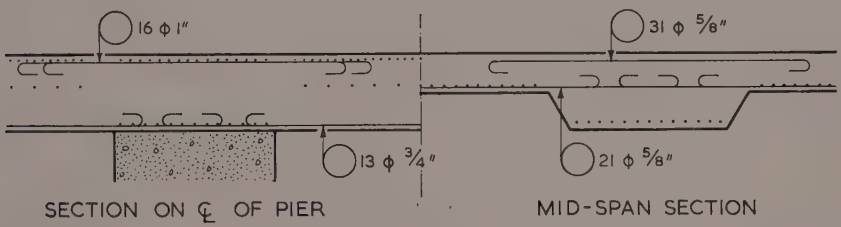


Fig. 16.24.

Moment in Strip B $= \pm\dfrac{WL}{72} = 118{,}000$ lb.-ft. on a width of 7 ft. $10\frac{1}{2}$ in.

Minimum d for resistance moment of $60bd^2 = \sqrt{\dfrac{118{,}000 \times 12}{94{\cdot}5 \times 60}} =$ say 16 in.

Negative A_{st}, at 12,000 lb. per sq. in. = thirty-one $\frac{5}{8}$-in. diam. bars.
Positive A_{st}, at 18,000 lb. per. sq. in. = twenty-one $\frac{5}{8}$-in. diam. bars.

The main reinforcement is shown in *Fig.* 16.24.

Floors on Piles with Specified Set

Several proprietary forms of piling are on the market, some of which claim to have high bearing capacities. To arrive at definite figures three different sizes of driven reinforced concrete piles will be considered.

10 in. by 10 in. up to 20 ft. long carrying 30 tons
12 in. by 12 in. „ „ 30 ft. „ „ 45 „
14 in. by 14 in. „ „ 40 ft. „ „ 60 „ .

In all normal cases the smallest diameter of pile consistent with length would be used as this means more piles, shorter spans and a thinner floor.

In the following it is assumed that there is a very solid bearing stratum in which the piles are certain to reach their specified set.

Using a flat-slab floor the maximum spacings of the piles, allowing for the self-weight of the floor will be as follows:

Depth of tank in feet	10-in. by 10-in. pile carrying 30 tons	12-in by 12-in. pile carrying 45 tons	14-in. by 14-in. pile carrying 60 tons
10	9 ft. 3 in. × 9 ft. 3 in.	11 ft. 3 in. × 11 ft. 3 in.	12 ft. 9. in. × 12 ft. 9 in.
20	6 ft. 11 in. × 6 ft. 11 in.	8 ft. 5 in. × 8 ft. 5 in.	9 ft. 8 in. × 9 ft. 8 in.
30	5 ft. 9 in. × 5 ft. 9 in.	7 ft. 0 in. × 7 ft. 0 in.	8 ft. 0 in. × 8 ft. 0 in.
40	5 ft. 0 in. × 5 ft. 0 in.	6 ft. 1 in. × 6 ft. 1 in.	7 ft. 0 in. × 7 ft. 0 in.

The natural type of flat-slab to use is the original elementary one without drops and without column capitals. Comparing this with a flat-slab floor in a warehouse, in the present case there is no stiffness from a column below as the top ends of the piles stand in soft ground and are fully stressed with their vertical load. Against this there is no possibility of uneven loading from panel to panel. In a building the columns are (or should be) spaced within an accuracy of 1 in. in a length of 20 ft. The head of a driven pile may easily wander 2 in. in any direction and a nominal distance of 6 ft. may easily be as little as 5 ft. 8 in. or as much as 6 ft. 4 in. Safe moments for the internal panels of this type of construction are, for Strip A: $-\frac{1}{18}WL$ and $+\frac{1}{24}WL$; and for Strip B: $\pm\frac{1}{60}WL$.

The section of Strip A is $\frac{1}{2}L$ in width, and if d is the overall thickness of the slab and a resistance moment of $60bd^2$ is to be assumed, then

$$\text{maximum unit } M = \frac{WL}{18} \times \frac{2}{L} \text{ lb.-ft. per foot.}$$

$$= \frac{WL}{18} \times \frac{2}{L} \times 12 \text{ lb.-in. per foot} = 60 \times 12 \times d^2.$$

$$\text{Minimum} \qquad d = \sqrt{\frac{W}{540}} = \frac{\sqrt{W}}{23{\cdot}2}.$$

For a 10-in. by 10-in. pile, $W = 30$ tons $= 67{,}200$ lb.

$$d = \frac{\sqrt{W}}{23{\cdot}2} = \frac{\sqrt{67{,}200}}{23{\cdot}2} = \frac{259}{23{\cdot}2} = 11{\cdot}25 \text{ in.; say } 12 \text{ in.}$$

For a 12-in. by 12-in. pile, $d = \dfrac{\sqrt{100{,}800}}{23{\cdot}2} =$ say 14 in.

For a 14-in. by 14-in. pile, $d = \dfrac{\sqrt{134{,}400}}{23{\cdot}2} =$ say 16 in.

Punching shear strength of 10-in. by 10-in. pile at 200 lb. per sq in.

$$= \frac{4 \times 10 \times 12 \times 200}{2{,}240} = 43 \text{ tons.}$$

Punching shear strength of a 14-in. by 14-in. pile $= \frac{4 \times 14 \times 16 \times 200}{2{,}240} = 80$ tons.

Beam shear strength on 45 deg. line for 14-in. by 14-in. pile (see *Fig.* 16.25) at 100 lb. per sq. in. $= \frac{4 \times 41{\cdot}5 \times 0{\cdot}88 \times 13{\cdot}75 \times 100}{2{,}240} = 90$ tons.

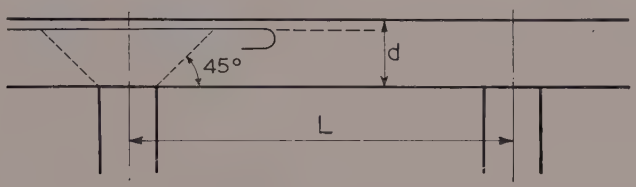


Fig. 16.25.

For a tank 30 ft. deep with 14-in. by 14-in. piles spaced at 8-ft. by 8-ft. centres and with a 16-in. slab, $W = 60$ tons $= 134{,}400$ lb.

$$WL = 134{,}400 \times 8 \text{ lb.-ft.} = 12{,}900{,}000 \text{ lb.-in.}$$

Strip A: $-\frac{WL}{18} = -717{,}000$ lb.-in. $\quad +\frac{WL}{24} = +538{,}000$ lb.-in.

Strip B: $\pm\frac{WL}{60} = \pm 215{,}000$ lb.-in.

A_{st} over pile $= \frac{717{,}000}{12{,}000 \times 0{\cdot}88 \times 13{\cdot}75} = 4{\cdot}96$ sq. in.

A_{st} in Strip A at midspan $= \frac{538{,}000}{18{,}000 \times 0{\cdot}88 \times 13{\cdot}75} = 2{\cdot}47$ sq. in.

A_{st} in top of Strip B $= \frac{215{,}000}{12{,}000 \times 0{\cdot}88 \times 13{\cdot}75} = 1{\cdot}48$ sq. in.

A_{st} in strip B at midspan $= \frac{215{,}000}{18{,}000 \times 0{\cdot}88 \times 13{\cdot}75} = 0{\cdot}99$ sq. in.

The minimum area of steel in a strip 48 in. wide is 0·3 per cent. of 48 × 16, that is 2·31 sq. in.

A proposed arrangement of reinforcement is shown in *Fig.* 16.26. This needs 0·64 cwt. of steel per sq. yd.

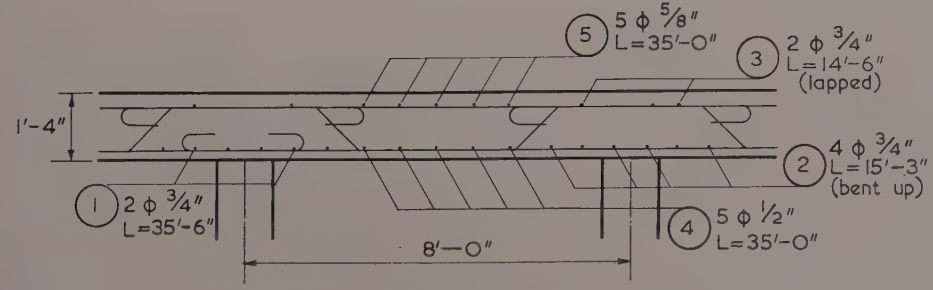


Fig. 16.26.

Floor on Piles—Uncertain Sets

In patchy ground where the piles may or may not reach their specified set it is advisable to adopt an arrangement which can cope with these conditions. A simple form is the slab of uniform thickness shown in *Fig.* 16.27. If a reservoir is 20 ft. deep the load per sq. ft. is about 1,400 lb. or 0·63 tons.

If the piles exceed 20 ft. but do not exceed 30 ft. in length, 12-in. by 12-in. piles carrying a maximum load of 45 tons may be used. Taking the view that a beam may be three times as wide as the column on which it is supported a ' beam ' 3 ft. wide can be imagined running over each row of piles, and a ' slab ' spanning from ' beam ' to ' beam '. If the span of the slab is a ft. then the reversed moment in the slab is $\frac{1}{12}wa^2$.

The load per foot run on the ' beam ' is wa and if the piles in the row are spaced at l-ft. centres, then the reversed moment in the ' beam ' is $\frac{1}{12}wal^2$ or $\frac{1}{12}wal^2 \times \frac{1}{3}$ per foot width of beam. Accepting the idea that the thickness of the slab depends on the allowable tension in the concrete, the minimum floor thickness will be

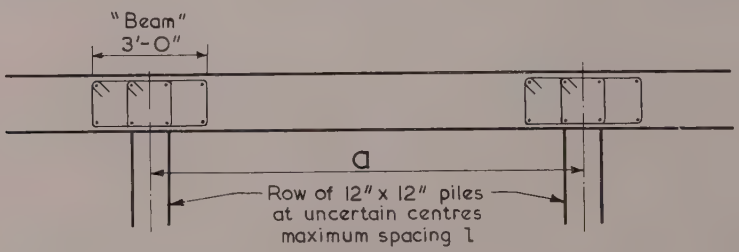


Fig. 16.27.

obtained when these two unit moments are equal, that is when $\frac{1}{12}wa^2 = \frac{1}{12}wal^2 \times \frac{1}{3}$, or $a = \frac{1}{3}l^2$. As the pile can only take a maximum of 45 tons,

$$al = \frac{45}{0{\cdot}63} = 71{\cdot}5 \text{ sq. ft. or } a = \frac{71{\cdot}5}{l} = \frac{l^2}{3}.$$

$l^3 = 215$ or $l = 6$ ft. very nearly (maximum) and $a = 12$ ft. say.
The unit reversed moment in the ' slab ' is $1{,}400 \times 12^2 \times \frac{12}{12} = 202{,}000$ lb.-in.
Loading on ' beam ' $= 12 \times 1{,}400 = 16{,}800$ lb. per foot.
Reversed $M = 16{,}800 \times 6^2 \times \frac{12}{12} = 605{,}000$ lb.-in. on 3-ft. width.

Minimum d for resistance moment of $60bd^2 = \sqrt{\frac{202{,}000}{60 \times 12}} = 16{\cdot}75$ in.; say 17 in.

A_{st} for reversed moment $= \frac{202{,}000}{12{,}000 \times 0{\cdot}86 \times 15} = 1{\cdot}28$ sq. in.

A_{st} for positive moment of $\frac{1}{24}wa^2 = \frac{101{,}000}{18{,}000 \times 0{\cdot}88 \times 15} = 0{\cdot}425$ sq. in.

Top steel in ' beam ' $= \frac{605{,}000}{12{,}000 \times 0{\cdot}86 \times 14{\cdot}25} = 4{\cdot}13$ sq. in.

Bottom steel in ' beam ' $= \frac{303{,}000}{18{,}000 \times 0{\cdot}88 \times 14{\cdot}25} = 1{\cdot}35$ sq. in.

The ' beam ' carries a load of 7·5 tons per foot. If the piles reach their specified set then they are spaced at 6-ft. centres. If they fail to do so the spacing is reduced.

The main bars in the ' beam ' are in long lengths hooked at both ends and lapped 40 diameters.

The minimum distribution steel required is 0·3 per cent. of 12 in. by 17 in. = 0·61 sq. in. per foot, which is more than the steel in the bottom of the slab. A suitable arrangement of steel is shown in *Fig.* 16.28. In most cases the

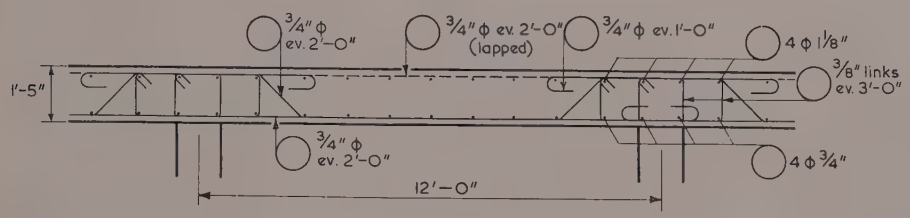


Fig. 16.28.

steel would have to be increased to take direct tension from the pressure on the walls; the drawing shows only the minimum amount.

$$\text{Punching shear stress} = \frac{45 \times 2{,}240}{4 \times 12 \times 17} = 123 \text{ lb. per sq. in.}$$

For a shallow reservoir on piles it might be better to keep the slab thickness to 9 in. and make an actual beam as shown in *Fig.* 16.29. This design is only practicable

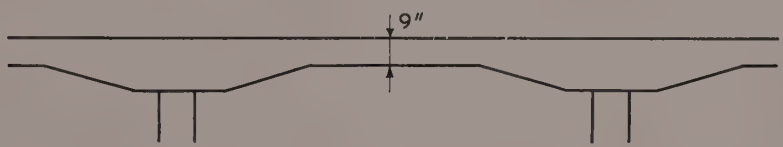


Fig. 16.29.

when the piles can be kept within an inch of their true alignment. The design shown in *Fig.* 16.28 will cope with normal variations but if the piles tend to wander more than 2 in. the steel (and particularly the bottom steel) should be increased.

Drainage

To secure efficient drainage from the roof, it is usual to slope the slab from the centre-line to the edges and to lay coarse gravel over the area before placing the selected back-filling on top. In some cases 3-in. agricultural pipes have been laid on the roof with the same object.

Hand-packed rubble filling or dry hardcore outside the wall is advisable for good construction unless the excavation proves to be thoroughly dry. Open-jointed land drains laid in coarse shingle, or French drains, may be laid in a trench at the back of the footing to remove the water which comes through the rubble and the discharge from the sloped roof of a covered tank. When the concrete of the footing slab has been placed in direct contact with the edge of the excavation it is better to arrange the land drains on the projection of the slab at the back of the wall. The pipes may be laid in lengths of about 30 ft. with a summit at the middle of each length. The required gradient is easily preserved by laying a sloping bed of weak concrete on the upper surface of the foundation slab. At the valley between each pair of lengths, 6-in. tile drains lead to a sump.

If there is a possibility that ground water may find its way into the subsoil immediately beneath the underside of the floor of the tank it is essential that a system of drainage is provided. A common arrangement is to have 3-in. or 4-in. land drains at 10-ft. or 12-ft. centres connecting to a larger drain. These are all laid with open joints in trenches filled with clinker, ballast, crushed rubble, etc. The preservation of a uniform grade is essential in order to prevent the deposit of silt in the pipes. Their depths are often limited by the available outfall. The main collector and the drain outside the footing of the wall, which takes away the water from the rubble packed behind the wall, may be connected to a manhole outside the tank. Leakage through the floor will be indicated by an increased flow through this manhole. The scour pipe leading from the sump in the bottom of the tank is also connected to this manhole. Another method of checking the level of the subsoil water is by inserting open-ended rainwater pipes in the columns of covered reservoirs. It is difficult to give a rule for determining the diameter required for the subsoil drains as this depends on the quantity of water met with in the excavation, the available fall, the nature of the ground, and the spacing of the pipes.

In some cases a pump has been installed above the sump to keep down the ground-water level and avoid the risk of the tank floating when it is empty. This, of course, is an expensive method of solving the problem, and should be avoided unless the tank is to be emptied frequently.

The concrete lining of the reservoir illustrated in *Fig.* 16.30 was laid in slabs 25 ft. square with drainage channels under most of the joints between the slabs in the sides and bottom, thus providing adequate escape for seepage should the joints leak. A copper water-stop and asphalt filler were used in the joints. Before the concrete lining was placed a trench was excavated along the line where the drained joints were to be placed, and a concrete rib or footing was cast, in the top of which was a semicircular channel. Seepage has access to this channel through grooves on each side spaced at 2-ft. centres. Over this trough a series of 17-in. by 24-in. precast reinforced concrete plates, 3 in. in thickness, were laid, these plates being intended as a covering for the channel and as a support for the edges of the lining slabs. The plates over the drain were left without bond to the concrete below to facilitate movement without cracking. The tops of the drainage channel covers were placed flush with the formation for the concrete lining, which was then concreted in the usual manner with the construction joints lying over the plates on the centre-line of the drainage gallery. The drainage galleries discharge into two 8-in. cast-iron collecting pipes at the centre of the reservoir, whence the water is conveyed under the reservoir embankment to the wash-out chamber. An 18-in. cast-iron scour pipe also leads from the centre of the reservoir to the wash-out chamber.

The division of the lining into slabs 25 ft. square with some reinforcement is expected to prevent shrinkage cracks in the slabs.

The importance of an adequate underdrainage system in the construction of a reservoir with partition walls cannot be too highly stressed. An extremely interesting account (' Engineering News-Record,' April 11, 1929) of the failure of a cantilever partition wall in a filtration plant built in 1915 in the United States shows how destructive unbalanced uplift can be. In this instance two coagulation basins, each 317 ft. long and 232 ft. wide, were separated by a division wall of the cantilever type 317 ft. long. This wall and its base (*Fig.* 16.31) formed an inverted T-shape, the base being 13 ft. 6 in. in width at one end, and at the other 19 ft. 6 in. wide and $16\frac{3}{4}$ in. thick at its outer edges, increasing to 2 ft. 3 in in thickness at the

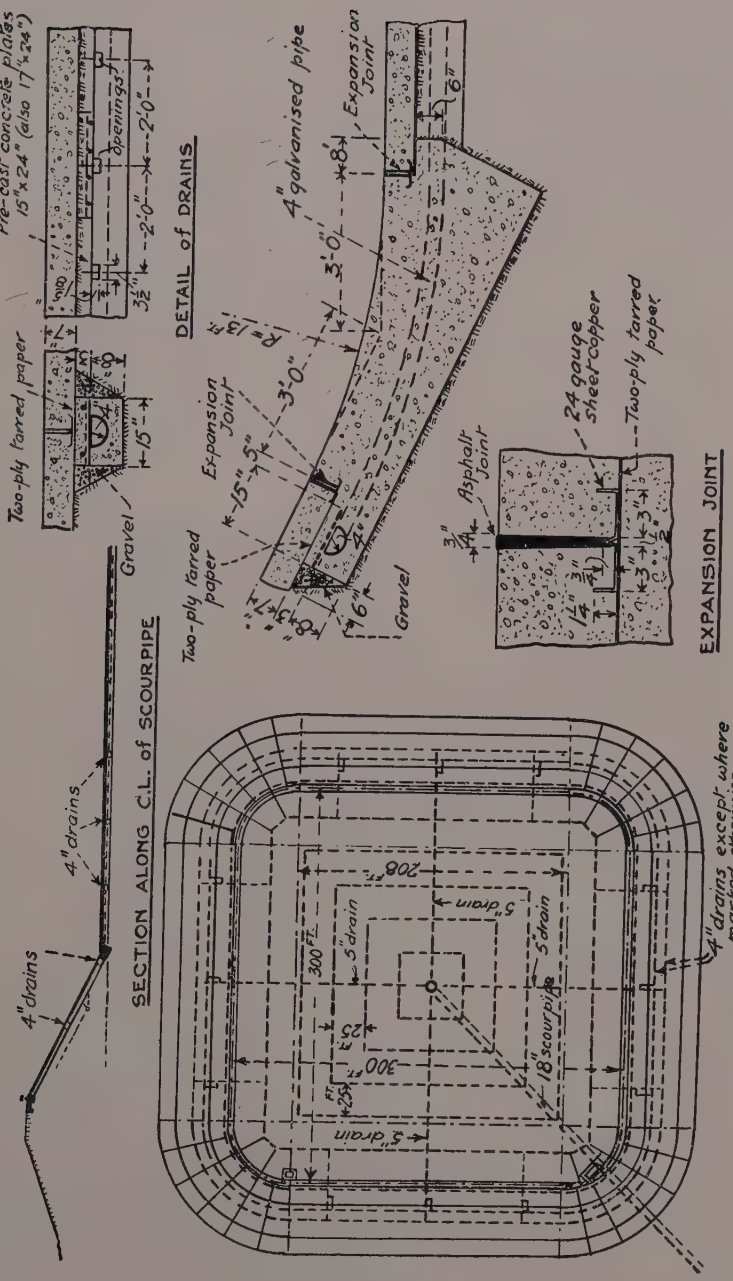

Pre-cast concrete plates 15"x24" (also 17"x24")
Openings
Two-ply tarred paper
Gravel
SECTION ALONG C.L. of SCOURPIPE
4" drains
4" drains
DETAIL of DRAINS
4" galvanised pipe
Expansion Joint
Expansion Joint
Two-ply tarred paper
Gravel
Asphalt Joint
24 gauge sheet copper
Two-ply tarred paper
EXPANSION JOINT
208 ft.
300 ft.
5" drain
5" drain
5" drain
5" drain
18" scourpipe
4" drains except where marked otherwise.
PLAN

Fig. 16.30.

wall. The latter was 14 ft. high, measured from the top of the base, at one end, and 19 ft. 6 in. at the other, 12 in. thick at the top, and battered on both faces so that the bottom thickness was nearly 2 ft. 6 in. The stem of the wall projected 2 ft. below the base to provide anchorage for the vertical bars in the wall and to give increased resistance against sliding. One-inch square twisted bars at $5\frac{1}{2}$-in. centres were used as vertical reinforcement in the lower part of the wall, and at higher levels the spacing of the bars was increased to 11 in. The floor was of plain concrete, 6 in. thick, and the formation what is locally known as 'rotten rock.' When stripped this required the use of picks and mattocks to excavate the foundation trenches. The excavation for the south wall stood vertically for more than a year, but on long exposure the material becomes soft. It is reported that all the bottom

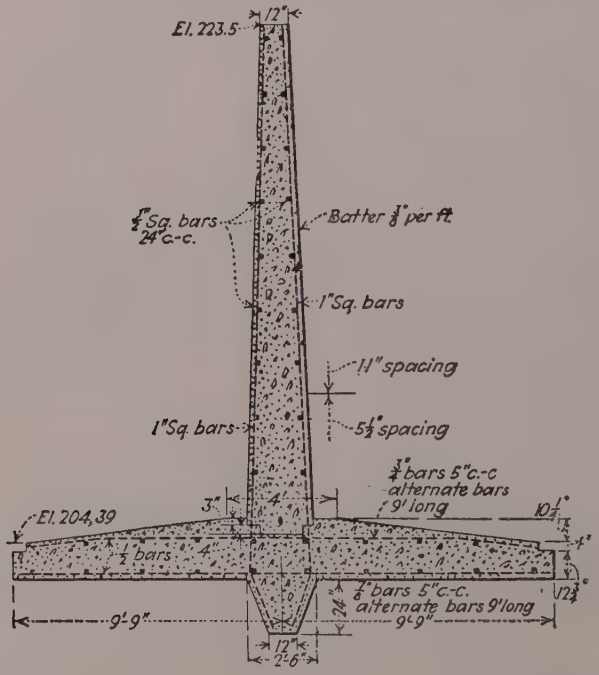


Fig. 16.31.

of the excavation was covered with concrete while the formation was still firm and hard. At the time of the failure one basin had been emptied, and after some time small streams of water were noticed spouting up at the joint between the wall footing and the floor slab of this basin. The latter was then pushed upward and the wall started to move. This movement started at the joint between two sections, and was followed by a general movement of the sections of the wall. The 12-in. by $\frac{1}{4}$-in. expansion plates, which extended 6 in. into each section of wall at each joint, were pulled out from one end of each section but remained fast in the other. A length of over 240 ft. of the wall was shifted and some sections, 32 ft. in length, were carried a distance of 60 ft. The drawing (*Fig.* 16.32) shows the final positions of the broken sections of the wall. Sections 3 and 4, which were unparted, are shown in *Fig.* 16.33.

In connection with the construction of some very large reservoirs of the Paris water supply, particular precautions were taken against settlement of the floors

due to leakage. The floors were made of arched cross-section supported on walls so that it is possible to walk under the reservoirs. Provision was made for draining away any leaking water and preventing it percolating into the ground and causing settlement of the piers.

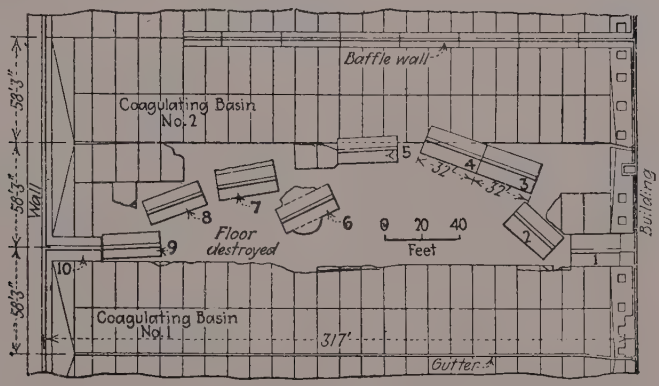


Fig. 16.32.

Fig. 16.33.

In countries subject to severe rainstorms it must be remembered that drains intended to lead water away from the subsoil may reverse under freak conditions and blow up the floor of the tank.

Pipe Connection through Tank Floor

To connect an inlet pipe passing vertically through a two-course floor, a short length of flanged pipe may be fixed vertically with its flange at the level of the top of the lower course. A 3-ft. square sheet of 20-gauge copper with a central opening for the pipe is then fitted over the pipe, and the rising pipe and short length of inlet bolted together through their flanges. Before laying the top course, a 1-in. asphalt joint is formed around the rising main. This type of joint has proved very satisfactory.

CHAPTER SEVENTEEN

RESERVOIR AND TANK WALLS SUPPORTED ON PILES

Wall to Circular Tank with Slip Joint

If an efficient frictionless slip joint (assuming that such a thing exists) is provided between the base of the wall and the floor, the piles under the wall will carry the weight of the wall itself plus a load from the floor depending on the span L (*Fig.* 17.1).

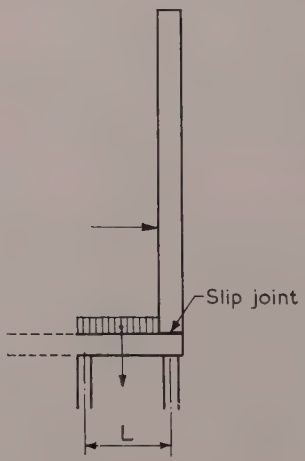


Fig. 17.1.

There may also be loads on top of the wall or loads carried on brackets halfway down the wall, or additional loads (other than water pressure), such as columns supporting the roof, on the floor. If, as is usual, the excavation is banked up round the outside of the wall there may be some downward friction on the wall as this filling consolidates and settles. In any case conditions at the base of the wall will not affect the design of the main part of the floor.

Wall to Circular Tank with Restraint

When the wall is restrained at the base, in addition to the vertical load from the wall the supporting structure must also resist a restraint moment M_B and corresponding shear F_B at the base of the wall (*Fig.* 17.2). Unless raking piles are provided, the shear force F_B must be taken directly or obliquely across the floor to the opposite side of the tank and this will require additional steel and possibly an increase in thickness to keep the tensile stresses in the concrete within safe limits.

In *Fig.* 17.3 the edge of the tank floor in *Fig* 3.27 is shown in detail. The wall is 1 ft. 6 in. in thickness and is supported on a ring of piles P_1 driven on a radius of 43 ft. 3 in. Inside this ring is another ring of piles P_2 driven on a radius of 36 ft.

3 in. supporting the floor, which is also 1 ft. 6 in. in thickness. Considering a wedge-shaped section on plan (part of a sector of the whole tank) which is 1 ft. wide at the face of the wall, this section will be 0·853 ft. wide on the smaller radius. The area of floor is $0{\cdot}5(0{\cdot}853 + 1{\cdot}0) \times 6{\cdot}25$ or 5·8 sq. ft. and the total inclusive load on it

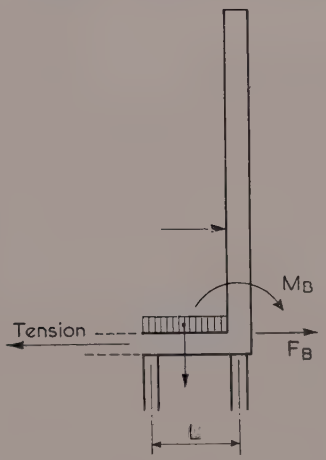


Fig. 17.2.

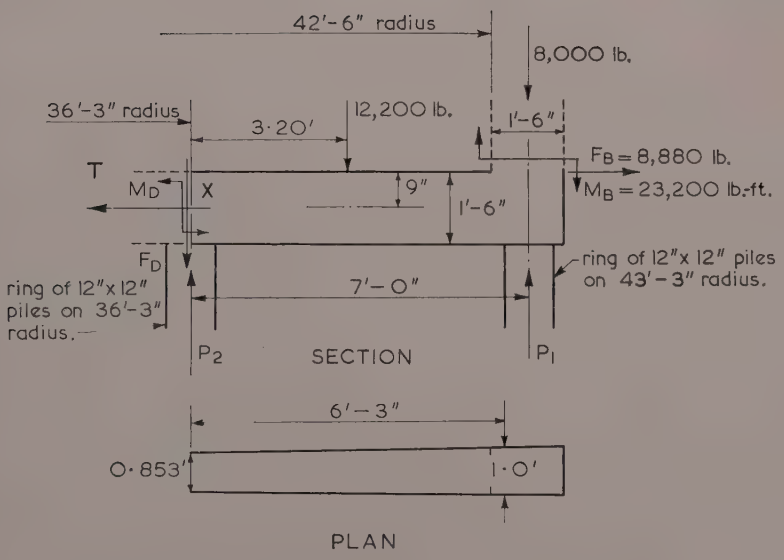


Fig. 17.3.

is 12,200 lb. The total inclusive load per foot run of wall is 8,000 lb. At the section over pile P_2 there will be a tension T, a shearing force F_D and a moment M_D which will depend on the spacing of piles under the central part of the floor and which has been calculated in this example as 8,710 lb.-ft. per foot run of section or 7,430 lb.-ft. on a width of 0·853 ft. The restraint moment M_B is 23,200 lb.-ft. per foot run of wall and the restraint shear F_B is 8,880 lb. per foot run.

Taking moments about point X (the point where T and F_D intersect):

$$P_1 \times 7{\cdot}0 = (12{,}200 \times 3{\cdot}20) + (8{,}880 \times 0{\cdot}75) + (8{,}000 \times 7{\cdot}0) + 23{,}200 - 7{,}430.$$

$$P_1 = 16{,}800 \text{ lb. per foot run of wall} = 7{\cdot}5 \text{ tons per foot run of wall.}$$

If the piles P_1 may carry 50 tons each, they may be spaced at 6-ft. 8-in. centres. If 48 piles are actually provided under the wall they are spaced at 5-ft. 8-in. centres and each pile carries 42·5 tons.

Care must always be taken to ensure that there is no uplift on the piles P_2. The total vertical load is $(12{,}200 + 8{,}000 + F_D)$ lb. Of this, 16,800 lb. is carried on P_1, leaving $(3{,}400 + F_D)$ lb. on P_2. The exact value of F_D depends on the arrangement of piles under the central part of the floor. If F_D is 6,000 lb., then piles P_2 carry 9,400 lb. per 0·853 ft. or 4·93 tons per foot. If there are 32 piles P_2 in this ring, they are spaced at 7-ft. 1-in. centres and each carries 35 tons.

Cantilevered Wall on Piles

A cantilevered wall of height H supported on vertical piles is shown in *Fig.* 17.4. The moment at the base of the wall is $\frac{1}{6}wH^3$ and the shearing force is $\frac{1}{2}wH^2$. This may be taken by tension in the floor, which may have to be strengthened to take it. To balance the moment of $\frac{1}{6}wH^3$ the value of B is given by $\frac{1}{2}wHB^2 = \frac{1}{6}wH^3$ or $B = \frac{H}{\sqrt{3}}$. Adopting a factor of safety of 1·5, then $B = \frac{H}{\sqrt{2}}$. This is generally not necessary as the inner pile carries load from the main part of the floor.

It may not be possible to transmit the tension across the floor to the opposite

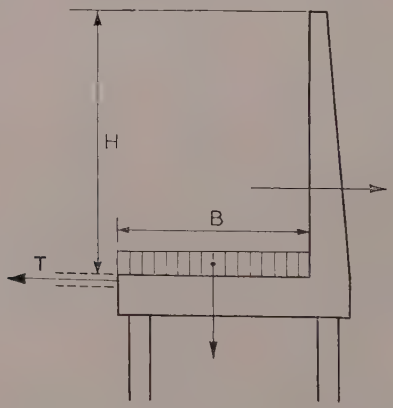


Fig. 17.4.

wall. Some reservoirs have a deep drainage trench that cuts the floor in two. In some cases the ground is good enough to carry a uniform load equal to the depth of the water but is not sufficiently good to support uneven loading from a cantilever wall. In this case the wall may be on piles and the centre of the floor may rest on the ground, with an expansion joint between them. If the floor cannot be utilized to tie back the base of the wall, raking piles are required to take the horizontal pressure. The first thing to be decided is the rake at which the piles may be driven. This depends on site conditions, on the length of pile and on the piling plant available. A rake of 1 in 3 is generally possible and sometimes 1 in 2. In

Fig. 17.5 both cases are shown for a pile carrying 50 tons. When driven at a rake of 1 in 3 this pile will carry a horizontal force of 15·85 tons, but before it can do this it must have a vertical load of 47·5 tons placed on its head. For every 1 ton of horizontal force 3 tons of vertical load must be supplied if the piles are driven to a rake of 1 in 3, or only 2 tons of vertical load if the piles rake 1 in 2.

A cantilevered slab wall 20 ft. high carried on piles and not tied back at the base is shown in *Fig.* 17.6. The base has been made equal to $\frac{20}{\sqrt{2}}$ or, say, 14 ft. The wall is carried on two rows of vertical piles V_1 and V_2 and a row of raking piles R_2. It is usually possible to drive these piles on a rake of 1 in 3. At first sight the arrangement in this figure looks satisfactory, the vertical load being shared by the two lines of vertical piles and the horizontal load being taken by the rakers, but a simple calculation shows it to be unsound. Produce the centre-line of the

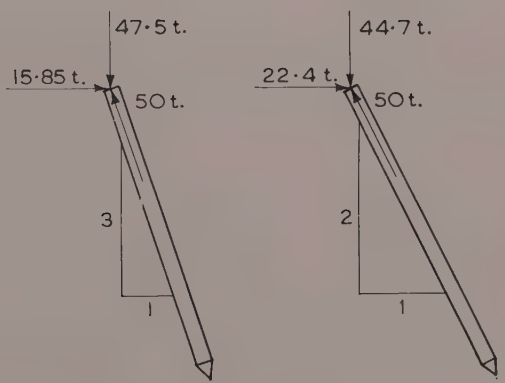


Fig. 17.5.

rakers upwards until this meets the centre-line of the inner vertical piles V_1. Taking moments about point O where these two lines intersect (the loads and water pressures per foot run of wall are written on the figure);

[(Load on Piles V_2) × 14 ft. 6 in.] + (12,500 × 33 ft. 8 in.)

= (8,140 × 7 ft. 6 in.) + (5,870 × 14 ft. 1 in.) + (17,500 × 6 ft.).

Load on Piles V_2 = −12,000 lb. per foot run of wall or a tension in the row of piles of 5·36 tons per foot run of wall. Whether this is satisfactory will depend on site conditions. Short piles driven through very soft ground and pulling up suddenly in something very hard offer little resistance to withdrawal. Long piles driven well into clay are difficult to draw as many contractors who have had to dismantle piled falsework well know. Most tank and reservoir walls have earth banking outside them, but modern specifications usually insist that the tank or reservoir is filled and tested before the banking is placed, so no relief can generally be counted on from this source.

Comparing *Fig.* 17.6 with *Fig.* 17.5 insufficient vertical load has been provided to hold down pile R_2 and the balance has to be provided by tension in pile V_2. If the raking piles in *Fig.* 17.6 can be driven at a rake of 1 in 2, then the dimension of 33 ft. 8 in. reduces to 19 ft. 2 in. and the load on piles V_2 changes from −12,000 lb. to +650 lb. per foot run of wall.

If it is not possible to drive at a rake of more than 1 in 3 and piles V_2 cannot

be relied on in tension, the design must be modified. Increasing the thickness of the wall will do little good, for the unbalanced moment about point O in *Fig.* 17.6 is 173,500 lb.-ft. and the weight of the wall would have to be increased by about 12,000 lb. per foot run which would necessitate providing an additional 80 sq. ft. of concrete per foot, equal to an additional width of 4 ft. on a wall 20 ft. high.

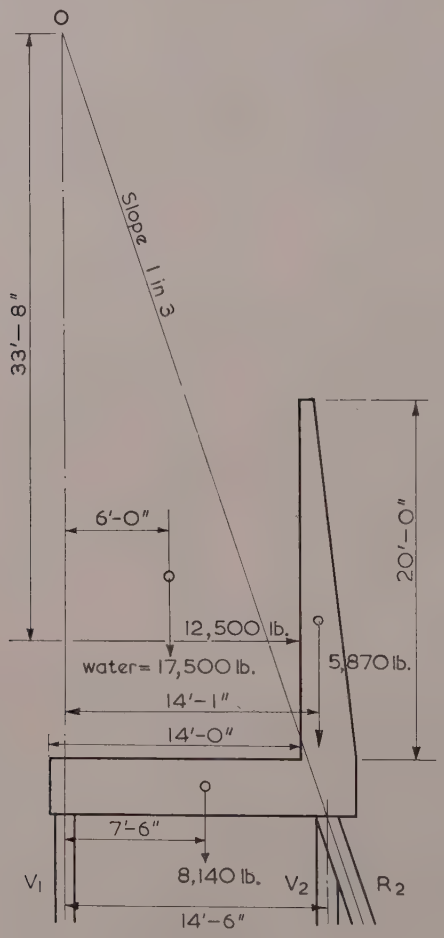


Fig. 17.6.

Little improvement follows if the base of the wall is increased as in *Fig.* 17.7, which shows the same wall as in *Fig.* 17.6 with the base increased in width from 14 ft. to 20 ft., as the height of point O rises 3 ft. for every 1 foot increase in the width of the base. The loading per foot run is written on the figure. Taking moments about point O:

$$[(\text{Load on Piles } V_2) \times 20{\cdot}5] + (12{,}500 \times 51{\cdot}7) = (11{,}000 \times 10{\cdot}5) + (5{,}870 \times 20{\cdot}1) + (25{,}000 \times 9{\cdot}0).$$

Load on Piles $V_2 = -9{,}150$ lb. per foot run of wall, which is still only 75 per cent. of the uplift given by the arrangement in *Fig.* 17.6. The total vertical load in

Fig. 17.7 is 41,870 lb., which is more than three times the horizontal pressure, but only part of this load comes on to the head of the raker R_2.

The problem can be solved by putting raking piles front and back as shown in *Fig.* 17.8. This disposition utilizes the whole weight of the wall. With this arrangement the loads on the piles cannot be found by simple statics and it is necessary to assume or to guess an arrangement of piles and then check the loading on them.

Assume that each set of piles consists of one vertical V_1, one raker R_1, one

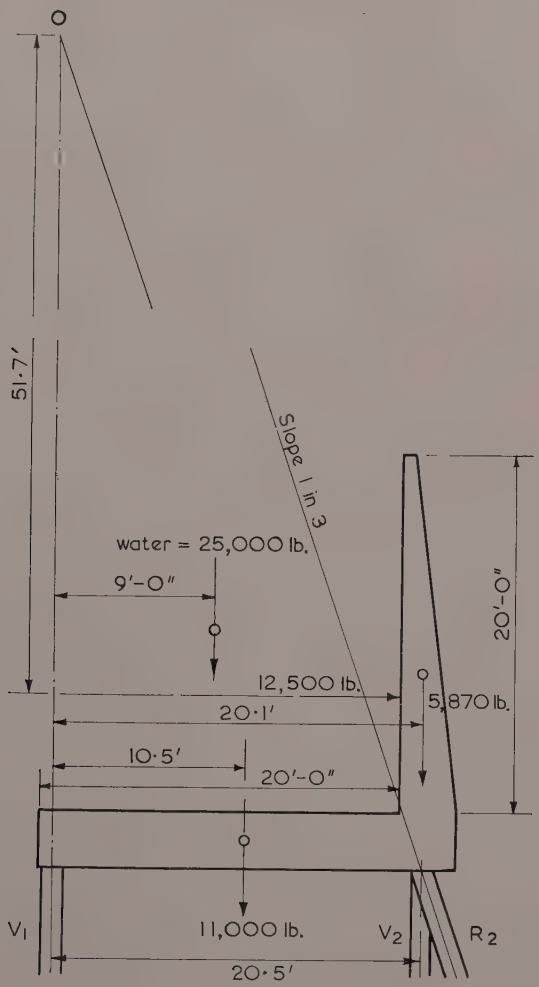


Fig. 17.7.

vertical V_2, and two rakers R_2, and that all the piles are of the same section and all are driven to the same depth, as shown in *Fig.* 17.8. If the vertical piles are 30 ft. in length the raking piles will be 31·6 ft. long, as they are driven at a rake of 1 in 3. Under the vertical and horizontal loads the wall will move and this movement may always be expressed as a vertical movement plus a horizontal movement plus a rotation about any point selected. If there are only two systems of piles as in *Fig.* 17.8, calculation can be saved by choosing the centre of rotation

at the point where the centroid lines of the two systems intersect (point X in *Fig.* 17.8). The line AXB is therefore drawn as being the centroid of the two vertical piles and the line CXD is drawn as the centroid of the three raking piles. These

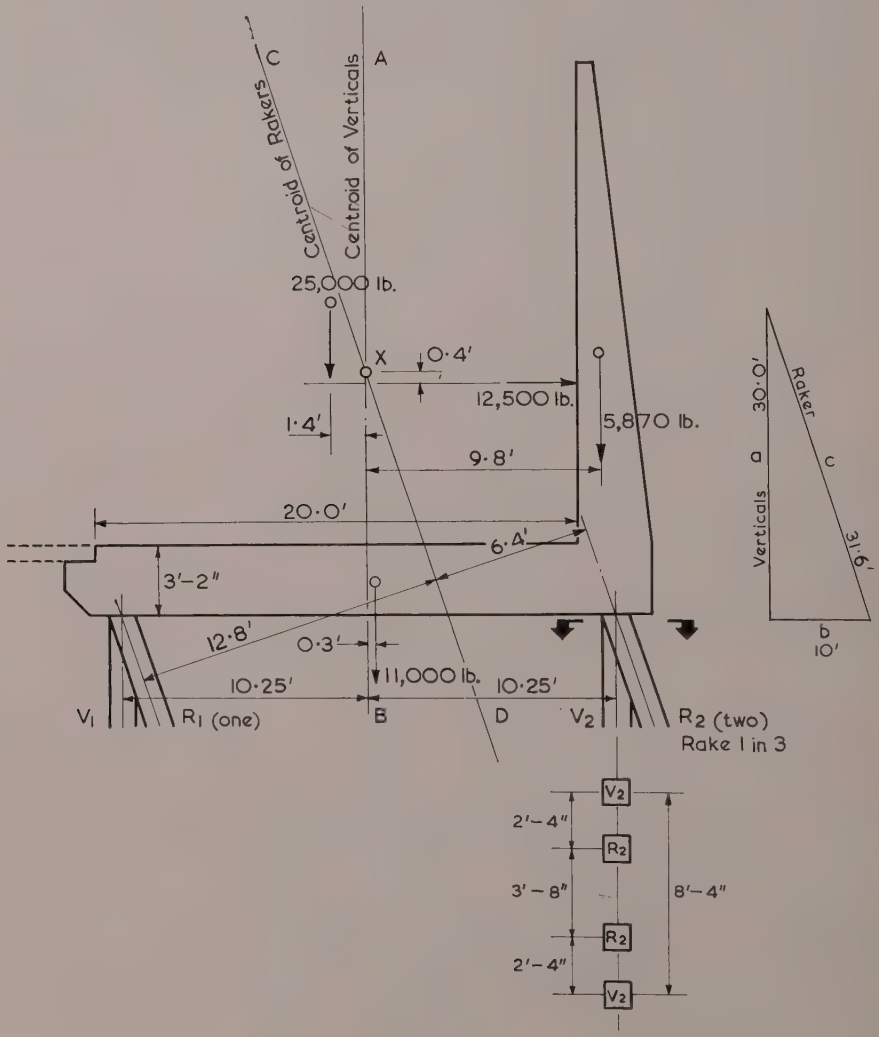


Fig. 17.8.

two lines intersect at the point X. Assume that the wall sinks an amount y, moves outward an amount x and rotates through a small angle θ clockwise about point X. Thrust in pile V_1

$$= x \times 0 \text{ (due to } x) + \frac{y(A)E_c}{30 \text{ ft.}} \text{ (due to } y) - \frac{10{\cdot}25\theta(A)E_c}{30} \text{ (due to } \theta),$$

where (A) is the cross-sectional area of the pile.
Thrust in pile R_1

$$= \frac{x \times 10(A)E_c}{31{\cdot}6^2} \text{ (due to } x) + \frac{y \times 30(A)E_c}{31{\cdot}6^2} \text{ (due to } y) - \frac{12{\cdot}8\theta(A)E_c}{31{\cdot}6} \text{ (due to } \theta).$$

$$\text{Thrust in pile } V_2 = \frac{y(A)E_c}{30} + \frac{10{\cdot}25\theta(A)E_c}{30}.$$

$$\text{Thrust in pile } R_2 = \frac{x \times 10(A)E_c}{31{\cdot}6^2} + \frac{y \times 30(A)E_c}{31{\cdot}6^2} + \frac{6{\cdot}4\theta(A)E_c}{31{\cdot}6}.$$

Since point X has been chosen as the intersection of the centroids, vertical or horizontal movement of this point causes no moment about X. Also rotation about X causes no net vertical or horizontal thrust.

$$\text{Total vertical reaction} = \left[\frac{2y}{30} + \left(\frac{3x \times 10}{31{\cdot}6^2} + \frac{3y \times 30}{31{\cdot}6^2}\right)\frac{30}{31{\cdot}6}\right] \times (A)E_c$$
$$= (0{\cdot}0285x + 0{\cdot}1522y) \times (A)E_c.$$

$$\text{Total horizontal reaction} = \left(\frac{3x \times 10}{31{\cdot}6^2} + \frac{3y \times 30}{31{\cdot}6^2}\right)\frac{10}{31{\cdot}6} \times (A)E_c$$
$$= (0{\cdot}0095x + 0{\cdot}0285y) \times (A)E_c.$$

Total moment of thrusts about X (anticlockwise)

$$= (A)E_c\left(\frac{2 \times 10{\cdot}25\theta \times 10{\cdot}25}{30} + \frac{12{\cdot}8\theta \times 12{\cdot}8}{31{\cdot}6} + \frac{2 \times 6{\cdot}4\theta \times 6{\cdot}4}{31{\cdot}6}\right) = 14{\cdot}81\theta(A)E_c.$$

Total vertical load = 25,000 + 11,000 + 5,870 = 41,870 lb.
Total horizontal load = 12,500 lb.
Total moment of loads about X (clockwise)

$$= (5{,}870 \times 9{\cdot}8) + (11{,}300 \times 0{\cdot}3) - (12{,}500 \times 0{\cdot}4) - (25{,}000 \times 1{\cdot}4)$$
$$= 20{,}800 \text{ lb.-ft.}$$

Equating the moments of the loads about point X to the moments of the thrusts about point X:

$$20{,}800 = 14{\cdot}81\theta(A)E_c \quad \text{or} \quad \theta(A)E_c = 1{,}400.$$

Equating the vertical loads and thrusts:

$$41{,}870 = (A)E_c(0{\cdot}0285x + 0{\cdot}1522y).$$

Equating the horizontal loads and thrusts:

$$12{,}500 = (A)E_c(0{\cdot}0095x + 0{\cdot}0285y).$$

Solving these last two equations:

$$x(A)E_c = 1{,}120{,}000 \quad \text{and} \quad y(A)E_c = 65{,}500.$$

Resubstituting these values for θ, x and y in the first expressions,

thrust in pile V_1 = 1,700 lb. R_1 = 12,600 lb.
V_2 = 2,660 lb. R_2 = 13,500 lb. (each.)

All these values are per foot run of wall. The results may be checked from the fact that the raking piles, being driven at a slope of 1 in 3, require 3 × 12,500 lb. vertical load to hold them down. Since the total weight is 41,870 lb., the sum of the loads on V_1 and V_2 should be (41,870 − 37,500) or 4,370 lb. If the maximum allowable load is 50 tons (112,000 lb.) per pile a pair of rakers and one vertical pile will be required every 8 ft. 4 in. under the toe of the wall. These could be

spaced as shown in *Fig.* 17.8. To comply with the assumptions on which the calculations are based, one raker and one vertical pile must be provided every 8 ft. 4 in. under the tail of the wall.

Under this loading pile V_2 carries only 8·33 × 2,660 lb. or only 10 tons, while V_1 carries only 6 tons and R_1 carries 47 tons.

When the tank is empty the calculated loads on the piles per foot run of wall are:

$$V_1 = +7{,}020 \text{ lb.} \qquad R_1 = -1{,}670 \text{ lb.}$$
$$V_2 = +9{,}830 \text{ lb.} \qquad R_2 = +835 \text{ lb. (each)}$$

The load on V_2 is 8·33 ft. × 9,830 = 36·5 tons and the load on V_1 is 26 tons.

If site conditions are such that R_1 can resist no tension then the wall (when

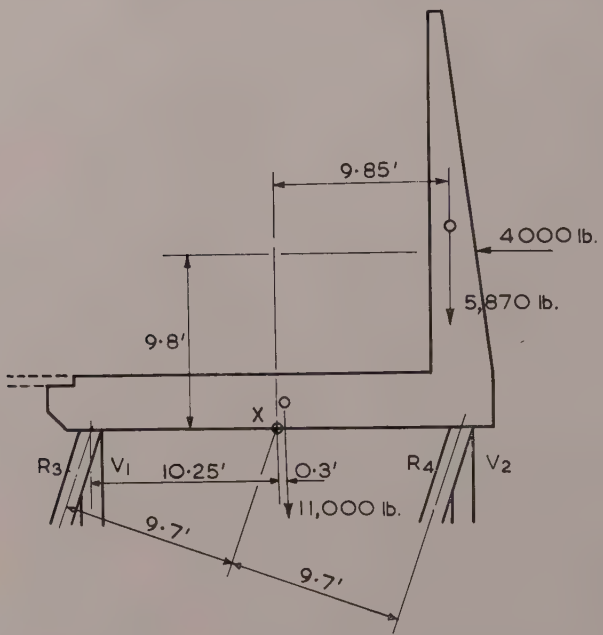


Fig. 17.9.

the tank is empty) is carried on the vertical piles alone and the load on V_2 rises to 11,400 lb. per foot or 42·3 tons.

If the tank is empty and the outward pressure of 12,500 lb. per foot is replaced by an inward pressure of 4,000 lb. per foot run of wall from earth banking, the loads on the piles per foot run of wall are:

$$V_1 = +12{,}990 \text{ lb.} \qquad R_1 = -5{,}920 \text{ lb.}$$
$$V_2 = +15{,}850 \text{ lb.} \qquad R_2 = -3{,}370 \text{ lb. (each)}$$

This would increase the load on pile V_2 to 59 tons.

If site conditions are such that R_1 and R_2 can resist no tension then normally the floor of the tank can be assumed to take the horizontal thrust of 4,000 lb. per foot, in which case $V_1 = +6{,}820$ lb. and $V_2 = 10{,}050$ lb. per foot run.

If the piles can take no tension and the floor can take no thrust, then additional rakers R_3 and R_4 could be provided as shown in *Fig.* 17.9. Under this arrange-

ment the loads on the piles per foot run of wall (assuming piles R_1 and R_2 are ineffective) are:

$$V_1 = +1{,}855 \text{ lb.} \qquad R_3 = +5{,}800 \text{ lb.}$$
$$V_2 = +3{,}005 \text{ lb.} \qquad R_4 = +6{,}840 \text{ lb.}$$

Alternatively the raker R_3 under the tail could be omitted and the wall thickened to provide extra weight, as shown in *Fig.* 17.10. With this arrangement, the

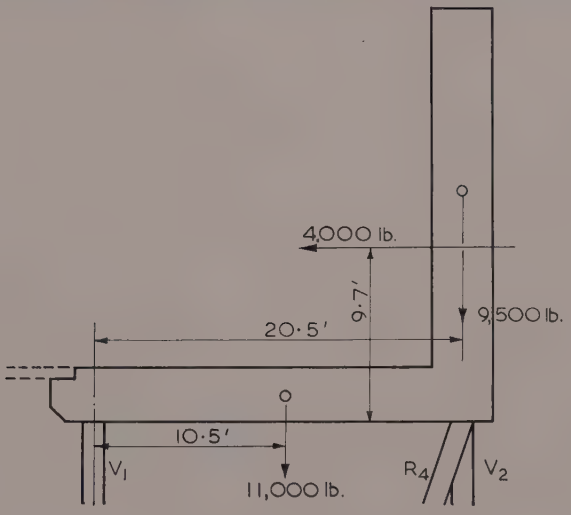


Fig. 17.10.

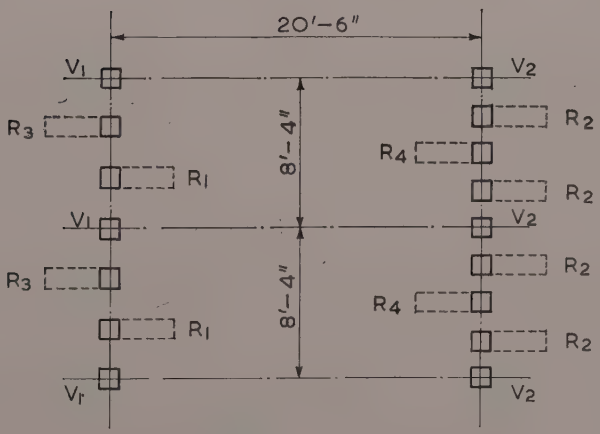


Fig. 17.11.

loads on the piles per foot run of wall (assuming piles R_1 and R_2 are ineffective) are:

$$V_1 = +7{,}240 \text{ lb.} \quad V_2 = +1{,}270 \text{ lb.} \quad R_4 = +12{,}640 \text{ lb.}$$

Therefore assuming that the wall can receive no support at all from the floor, that the piles are driven at a rake of only 1 in 3, that no tension is allowed in any pile, that earth banking will be placed after the tank has been filled and tested, and that the wall will not be thickened as shown in *Fig.* 17.10, the final arrangement of piles (assuming each can support 50 tons) is as shown in plan in *Fig.* 17.11.

The fact that the pile heads are close together is not important as their toes are well spaced out.

When detailing the steel in the base of the wall the designer must remember to tie in the heads of the raking piles. In *Fig.* 17.8, pile R_1 carries 12,600 lb. per foot run of wall or 105,000 lb. per pile. The horizontal component of this is 33,200 lb., which at 18,000 lb. per sq. in. requires 1·85 sq. in. of steel. Two long hairpins of $\frac{7}{8}$-in. diam. bar would take care of this.

The base of the wall in *Fig.* 17.8 could act as a horizontal beam 38 in. wide and 276 in. deep. If suitably reinforced this could carry the water pressure of 12,500 lb. per foot over more than 100 ft. If the tank were not more than 150 ft. square all raking piles could be dispensed with and the base of the wall could be used as a square collar beam (always assuming that the tank floor cannot be used for tying across from wall to wall). Site conditions would have to be extraordinarily difficult to make such a design an economical proposition.

The bending moments and thrusts on the base of the wall in *Fig.* 17.8 are shown in *Fig.* 17.12. The critical section is X–X in *Fig.* 17.13. At this point there is

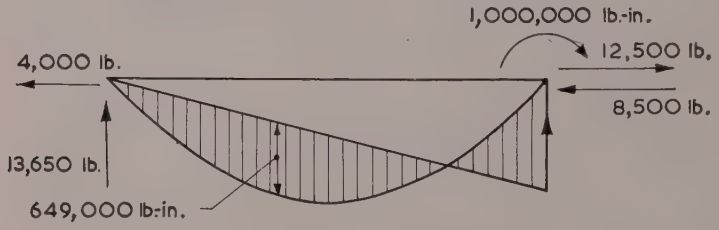


Fig. 17.12.

a moment of 1,000,000 lb.-in. from the wall plus a shear of 12,500 lb. Acting against this shear there is a horizontal component of 8,500 lb. per foot from the raking piles R_2, so that there is an unbalanced tension of at least 4,000 lb. applied at the top edge. Transferring this tension 19 in. to the centre-line of the section:

moment =	1,000,000 lb.-in.
+4,000 × 19 =	76,000 ,, ,,
	1,076,000 lb.-in.

As far as concrete stresses are concerned,

$$\text{area} = 12 \times 38 = 456 \text{ sq. in. and } Z = 12 \times 38^2 \times \tfrac{1}{6} = 2{,}890 \text{ in.}^3$$

Adding about 1 per cent. of steel will increase the area by about 14 per cent. and the value of Z by about 40 per cent.

The tensile stress in concrete is about $\frac{1{,}076{,}000}{2{,}890 \times 1{\cdot}4}$ =	266 lb. per sq. in.
$+\frac{4{,}000}{456 \times 1{\cdot}14}$ =	8 ,, ,, ,, ,,
	274 lb. per sq. in.

Area of top steel required, working at a stress of 12,000 lb. per sq. in.

$= \frac{1{,}076{,}000}{12{,}000 \times 0{\cdot}9 \times 35{\cdot}5}$ =	2·80 sq. in.
$+\frac{2{,}000}{12{,}000}$ =	0·17 ,, ,,
	2·97 sq. in.

$1\frac{1}{8}$-in. diam. bars at 4-in. centres provide 2·98 sq. in. On the one hand it may be argued that the shear comes on the top face of Section X–X while the raking piles are at the bottom, and the section should therefore be designed for the full tension of 12,500 lb. On the other hand those cracks in tanks that are not due to shrinkage or temperature stresses are due to poor arrangements of bars and insufficient grip lengths and are not due to high calculated structural stresses. With the bar arrangement shown in *Fig* 17.13, any practical designer would accept a calculated stress of 13,500 lb. per sq. in. and most would accept 16,000 lb. per sq. in. in the steel.

The method of calculating the loads on the piles shown in *Fig* 17.8 also gives the actual movement of the wall, always provided that everything is expressed in the same units. As the lengths of the piles have been given in feet it is easiest to work in pounds and feet throughout.

Assuming a 14-in. by 14-in. pile reinforced with four $1\frac{1}{8}$-in. diam. bars the area

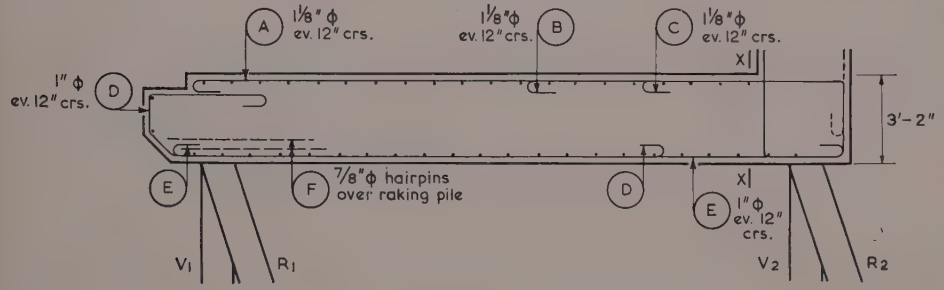


Fig. 17.13.

is [196 + (14 × 4)] sq. in. or 1·75 sq. ft. Since the pile group repeats at 8-ft. 4-in. centres the area $(A) = \frac{1{\cdot}75}{8{\cdot}33} = 0{\cdot}21$ sq. ft. per foot run of wall.

Assuming $E_c = 2{,}000{,}000 \times 144$ lb. per sq. ft.,

$$x = \frac{1{,}120{,}000}{(A)E_c} = \frac{1{,}120{,}000}{0{\cdot}21 \times 2{,}000{,}000 \times 144} \text{ ft.} = 0{\cdot}222 \text{ in.}$$

This is the outward movement of point X.

$$\theta = \frac{1{,}400}{(A)E_c} \text{ radians (clockwise)} = \frac{1{,}400}{0{\cdot}21 \times 2{,}000{,}000 \times 144} = \frac{1}{86{,}400} \text{ radians.}$$

As point X is about 100 in. above floor level, this causes a back-kick of

$$\frac{100}{86{,}400} \text{ in. at floor level} = 0{\cdot}0012 \text{ in.}$$

The vertical (downward) movement of point X is $y = \frac{65{,}500}{(A)E_c} = 0{\cdot}013$ in.

Therefore the slip joint where the main floor rests on the tail of the wall must be able to take up a movement of about $\frac{1}{4}$ in., assuming that the piles are 30 ft. long. If the piles are longer than 30 ft. the movement will be increased pro-rata.

The method of finding the thrusts in a combination of vertical and raking piles by assuming horizontal, vertical and rotational displacements of the wall may be

used for more complicated cases where piles of differing sizes and lengths are driven at different rakes.

If there are only two systems of piles as in the systems in *Figs.* 17.8 and 17.9 considered separately, point X can be found. However the two systems shown in these figures are combined there is no common intersection point. In such a case any point may be chosen as the reference point but the thrust in each pile must then be calculated for each of the three displacements and this total thrust must be included in each of the three equations of equilibrium. Each of these three equations will then include x, y and θ, and the complexity of the calculations increases. Such cases are easily dealt with on a double-elephant drawing board but are very cumbersome on the small page of a text-book.

One advantage of walls on piles is that the piles are usually sufficiently long and flexible to take up small horizontal displacements at their heads, and expansion or contraction joints are not required in walls of normal length.

CHAPTER EIGHTEEN

ELEVATED TANKS

The term ' elevated tank ' is here meant to apply to tanks where the height from ground level to floor level does not exceed about 20 ft. Above this height the tank is more properly described as a ' water tower '.

When the floor of a tank is only some 2 ft. or 3 ft. above foundation level it is generally cheaper to fill in with weak mass concrete or hardcore flushed with concrete. When the difference in level exceeds 6 ft. or 7 ft. a suspended floor will usually be cheaper in this country. In doubtful cases the designer must draw and price both alternatives. If a considerable amount of earth filling has to be disposed of on the site, this may be filled in between the sleeper walls or supporting columns, covered with 3 in. of mass concrete and used as shuttering for the underside of a suspended floor. The earth filling must be rammed sufficiently solidly to support the weight of the floor for a few hours but almost any type of soil, apart possibly from peat, is suitable for this.

Early Designs

In *Fig.* 18.1 alternative designs are shown for a tank 108 ft. in diameter to hold filter bed or similar material 5 ft. deep weighing 100 lb. per cu. ft. In the left-hand half of the figure a plain slab supported on circular sleeper walls is shown. This design is based on principles in general use prior to 1914. A 6-in. floor slab designed for a maximum moment of $\frac{1}{10}wl^2$ with stresses of 600 and 16,000 lb. per sq. in. in the concrete and steel and supported on 4-in. walls was quite usual practice. Taking the effective depth of a 6-in. slab as 4·75 in. it would be designed to provide a resistance moment of $95bd_1^2$ or $95 \times 12 \times 4{\cdot}75^2 = 25{,}800$ lb.-in. per foot width (present-day design would adopt a resistance moment of $60bd^2 = 60 \times 12 \times 6^2$ or 25,900 lb.-in.). If $w = (5 \times 100) + 75$ lb. and $\frac{1}{10}wl^2 = 25{,}800$, then $l = 6{\cdot}12$, say 6 ft. span. A 7-in. slab would span 7 ft. 3 in. In the figure sleeper walls are shown at 7-ft. 9-in. centres and a $7\frac{1}{2}$-in. slab would span this. The outermost wall could be kept back from the outside edge of the slab so that the cantilever moment over this wall was the same as the reversed moment over the inner walls.

Whether it was cheaper to use a thinner slab and more sleeper walls or a thicker slab and fewer sleeper walls would depend on site conditions and usually meant working out and pricing several alternatives.

After the introduction of flat-slab construction the floor could have been designed as a standard flat-slab using the Chicago Regulations (1918). This arrangement is shown in the right-hand side of *Fig.* 18.1. Under these regulations a 6-in. slab would span 9 ft. by 9 ft. as shown in the figure. An 8-in. slab would span 12 ft. 6 in. by 12 ft. 6 in. under an imposed load of 500 lb. per sq. ft. Again it might be necessary to draw and price designs with alternative column spacings.

If the tank were supported on piles carrying 40 tons each, a mesh-panel arrangement with the slab designed to carry a moment of $\frac{1}{30}W$ might have been used, as shown in *Fig.* 18.2. Other possible variations are indicated in Chapter 16.

Many of these early designs are open to severe criticism as few, if any, made

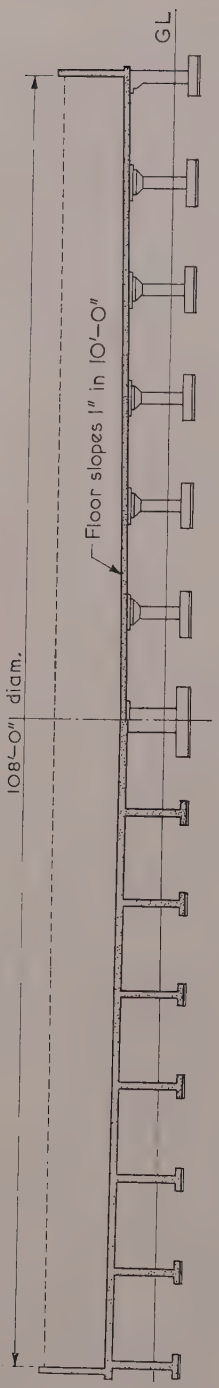


Fig. 18.1.

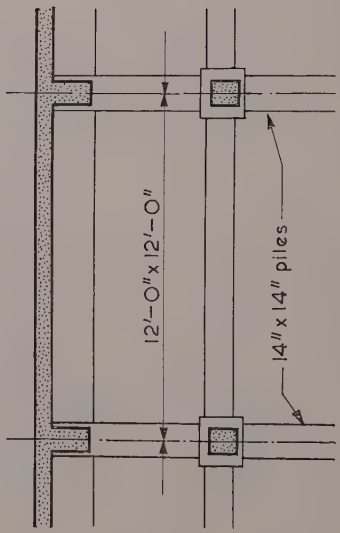


Fig. 18.2.

any provision for stresses due to temperature change. Since the floor is no longer in close contact with solid ground it must be subject to greater changes of temperature than a floor below ground level and possibly also subject to some shrinkage. Nevertheless these tanks generally gave satisfactory service and they cannot be lightly dismissed as ' old-fashioned '.

More Recent Designs

Few designers would now use a wall less than 6 in. in thickness and some authorities may insist on a minimum of 8-in. for a load-bearing wall. Working to the requirements of CP.2007:1960, the stress in the steel near the underside of the floor may be increased to 18,000 lb. per sq. in. if the floor is 9 in. or more in thickness.

The use of thicker supporting walls or larger supporting columns increases the temperature stresses and makes it difficult to design a suspended floor for a large tank less than 10 ft. above foundation level. If the tank is 100 ft. in diameter and if the floor is subject to a temperature range of ± 30 deg. F., the outside edge of the floor will move $\pm 30 \times 0{\cdot}000006 \times 50$ ft. $= \pm 0{\cdot}108$ in.

Circumferential walls, or columns supporting the floor (assuming that no rocking or sliding joints are provided) will be flexed as shown in *Fig.* 18.3, the

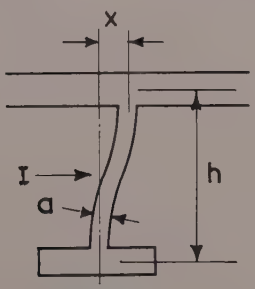


Fig. 18.3.

displacement x varying from zero at the centre of the tank to a maximum at the circumference. If the effective height of the column is h and its geometrical moment of inertia of cross-section is I then the primary moments caused by the displacement are $\frac{6EIx}{h^2}$. This value varies inversely as h^2 but directly as a^3, where a is the thickness of the wall or the width of the column.

The effect of temperature stresses on a tank near ground level may be shown by analysing the design in *Fig.* 18.4. This shows a tank 100 ft. in diameter, 5 ft. above the foundations carrying a 5-ft. depth of material weighing 100 lb. per cu. ft. and supported on five walls 8-in. in thickness, which are circular in plan and 11 ft., 33 ft., 55 ft., 77 ft. and 99 ft. in diameter, respectively. Neglecting temperature stresses the maximum moment in the slab is about

$$\tfrac{1}{10}wl^2 = \tfrac{1}{10} \times (500 + 150) \times 11^2 \times 12 = 94{,}300 \text{ lb.-in.}$$

With a moment of resistance of $60bd^2$ the minimum value of d is 11·45 in., say 12 in. The stresses may be analysed by considering a sector of the floor 1 ft. wide at the outermost wall, as shown in *Fig.* 18.4. This sector tapers to 0·778 ft. at the next wall and to 0·555 ft. at the next, as shown in the figure. The widths of slab at midspan are 0·889 ft. and 0·667 ft. as shown.

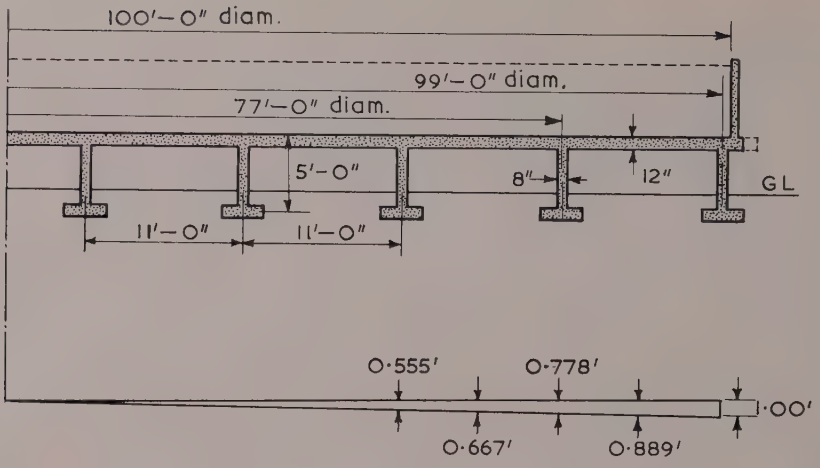
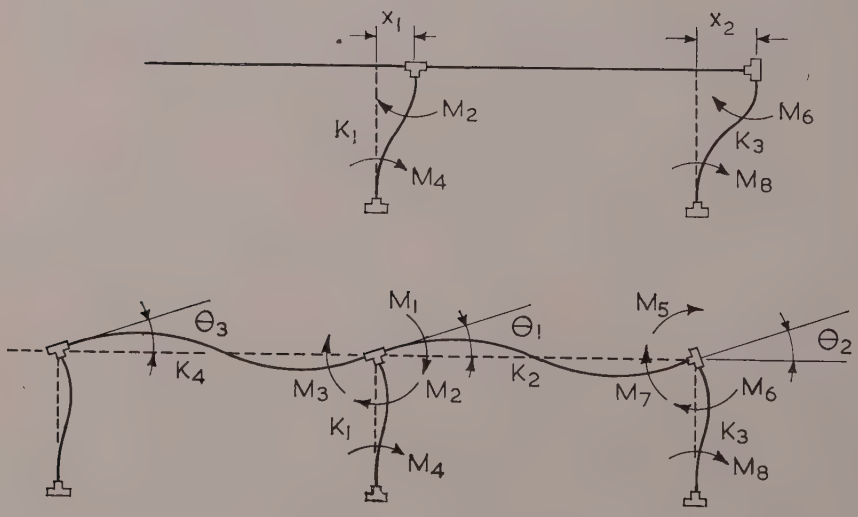


Fig. 18.4.



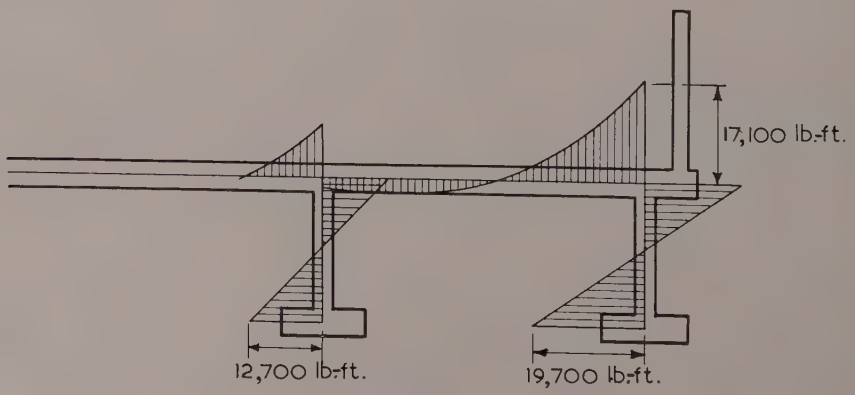


Fig. 18.5.

Assuming the lateral pressure from the contents is 30 lb. per sq. ft. per foot of depth and that the wall is designed to cantilever from the slab,

$$M_5 \text{ in } Fig.\ 18.5 = \tfrac{1}{6} \times 30 \times 5^3 = 625 \text{ lb.-ft.}$$

The fixed-ended moment in a strip of floor slab 1 ft. wide is $\frac{1}{12} \times (500 + 150) \times 11^2$ or 6,550 lb.-ft. For a final design special coefficients should be used for the slab as this tapers in plan, but for preliminary calculations the average width at midspan can be used and C_1 and C_2 can be taken as 4 and 2, respectively. Considering a temperature rise of 30 deg. F., $x_1 = 0{\cdot}00693$ ft. and $x_2 = 0{\cdot}00891$ ft.

The displacements are shown in *Fig.* 18.5. All the values needed to calculate the values of K and to tabulate the moments are now available.

$$M_1 + M_2 + M_3 = 0 \text{ and } M_5 + M_6 + M_7 = 0.$$

Assuming $\theta_3 = \frac{1}{2}\theta_2$,

$E\theta_1 = -139{,}500$ and $E\theta_2 = -324{,}300$ (both in pounds and foot units),

giving $M_7 = -17{,}100$ lb.-ft. and $M_8 = +19{,}700$ lb.-ft.

The moments in the outermost bay are shown in *Fig.* 18.5.

The calculated tensile stress in the top of the slab over the outer wall is very nearly

$$\frac{17{,}100 \times 12 \times 6}{12 \times 12^2} = 712 \text{ lb. per sq. in.}$$

or possibly 600 lb. per sq. in., allowing for the reinforcement, against a usual maximum of 300 lb. per sq. in.

The load on the outer wall is about 6,000 lb. per foot run with a moment of

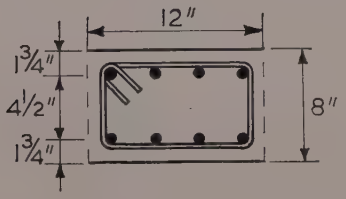


Fig. 18.6.

19,700 lb.-ft. at the base. This means that the eccentricity of loading on the foundation is about 3 ft. 3 in. and that the footing would have to be increased from 3 ft. to 8 ft. wide.

The wall must be designed to resist a moment of 19,700 lb.-ft. per foot run. Assuming vertical bars at 3-in. centres near both faces, the section per foot run of wall will be as shown in *Fig.* 18.6.

$$\frac{M}{bd^2} = \frac{19{,}700 \times 12}{12 \times 8^2} = 308 \text{ and } \frac{M}{bd_1^2} = \frac{19{,}700 \times 12}{12 \times 6{\cdot}25^2} = 504.$$

Both of these figures are very high.

Reverting to the old 'steel-beam' theory, ignoring the concrete and working to 25,000 lb. per sq. in.,

$A_{sc} = A_{st} = \dfrac{19{,}700 \times 12}{25{,}000 \times 4{\cdot}5} = 2{\cdot}1$ sq. in.; say four $\frac{7}{8}$-in. diam. bars both sides every foot run of wall (see *Fig.* 18.6). Not only is this an enormous amount of steel, but from *Fig.* 18.4. it is clear that there is not enough concrete to anchor these bars

into joints at the top and bottom of the wall, even if the diameter of the floor slab is extended as shown by the broken lines in *Fig.* 18.4. The preliminary design shown in *Fig.* 18.4 is therefore not workable.

If the permissible thickness of the walls may be reduced from 8 in. to 6 in. the primary moments caused by the horizontal movement will be reduced in the ratio of 8^3 to 6^3 or 1 to 0·422, but the lever-arm of the section, measured steel-to-steel will drop from 4·5 in. to 2·5 in. or in the ratio 1 to 0·555, giving little relief on balance.

If the height of the walls is increased in the ratio $\sqrt{2}$ to 1, that is if the foundations are deliberately taken down 2 ft. deeper, the primary moments due to temperature change are halved and the design would then be just workable.

With the introduction of rocking joints at the top and bottom of the outer walls as shown in *Fig.* 18.7 the primary moments vanish and the problem does not arise, but many engineers (including the author) dislike such joints.

Filling the empty spaces between the walls with earth and forming an earth

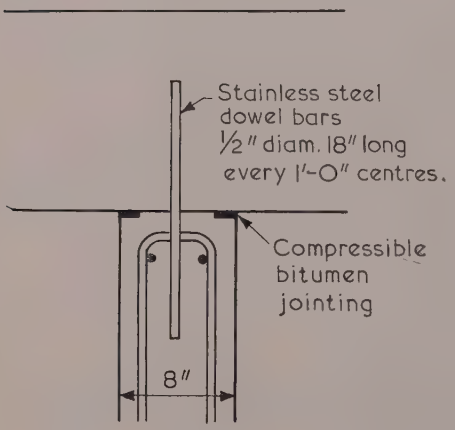


Fig. 18.7.

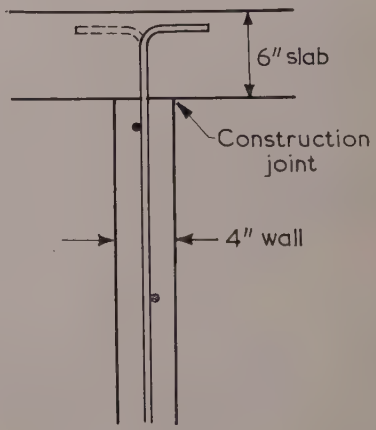


Fig. 18.8.

bank round the tank should decrease the range of temperature variation and decrease the primary temperature stresses pro-rata.

Going back to early designs, when few (if any) designers worried about temperature stresses, a 6-in. floor slab with 4-in. walls reinforced with nominal steel having very little end anchorage was quite usual. Such walls had a construction joint at the top and another at or near the bottom, and these joints opened and closed slightly as the wall was rocked backward and forward by expansion and contraction of the floor slab. In fact, if the actual detail at the top of the wall were as shown in *Fig.* 18.8 the designer had, unwittingly, introduced a rocking joint similar to the joint in *Fig.* 18.7.

Where the supporting walls ran radially, vertical cracks could develop in them which opened in summer and closed in winter. In one such structure the top construction joint sheared, but in most, if not all, cases the tanks continued to function.

With a long rectangular tank only a few feet above foundation level, the problem of movement due to temperature change can become acute and the possibility of cutting the tank into sections by expansion joints may have to be considered

as well as the provision of sliding or rocking joints. A detail of an expansion joint is given in *Fig.* 18.9, and in *Fig.* 18.10 a large shallow tank on piles with sliding

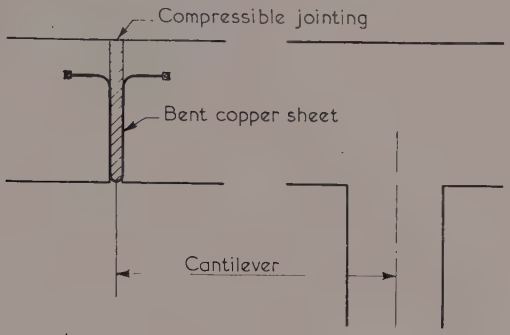


Fig. 18.9.

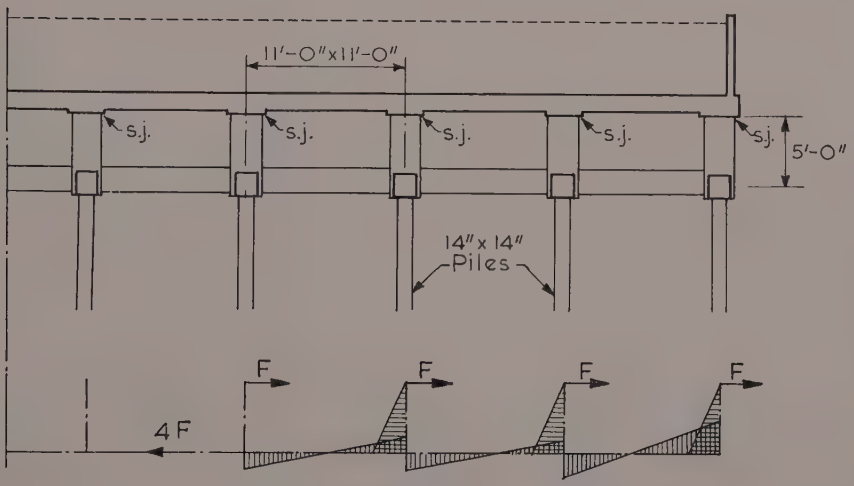


Fig. 18.10.

joints under the floor slab is shown; a proposed detail of the sliding joint is given in *Fig.* 18.11. The makers claim a coefficient of friction of 0·2 for some types of synthetic material sliding over stainless steel. Not only is there a heavy moment

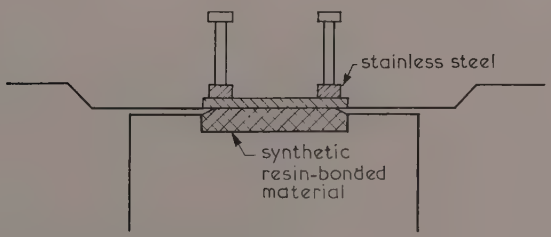


Fig. 18.11.

at the foot of the short columns in *Fig.* 18.10, but, the horizontal tension in the tie builds up to a high figure. Assuming each pile carries 40 tons and $\mu = 0{\cdot}2$, then $F = 8$ tons. The design depends entirely on the correct functioning of the joints. The lower pad may be bedded down on synthetic ' impact ' adhesive but must be set (or machined) dead level. The makers of certain types of built-up bearings state that a coefficient of friction as small as 0·05 may be expected with a useful life of 100 years. The author would introduce sliding joints of this type only as a last resort, if there were no alternative.

Columns under Elevated Tanks

The head of a column of height h in *Fig.* 18.12, supporting the floor of a tank at a distance R from the centre will be pushed out a distance $30 \times 0{\cdot}000006R$

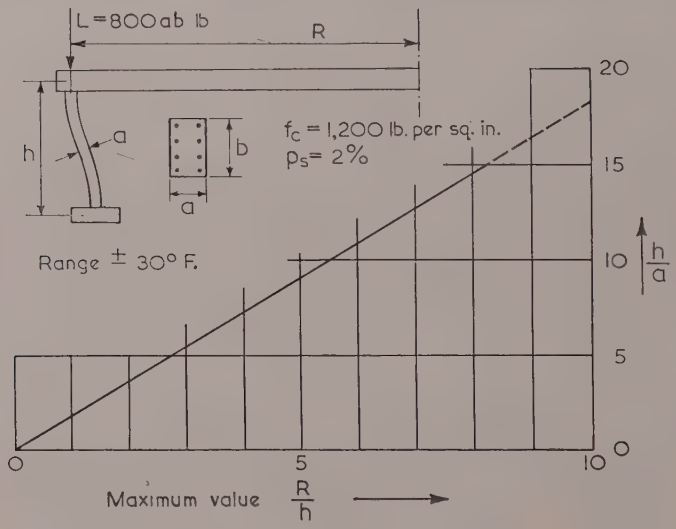


Fig. 18.12.

when the temperature of the tank floor rises 30 deg. F. If the floor is comparatively stiff and the footing slabs are reasonably wide, as a first approximation, the assumption may be made that the tangent slopes at top and bottom of the column remain vertical.

If b is the width of the column section and a the depth in the direction of movement, and if $E_c = 3{,}000{,}000$ lb. per sq. in.,

$$M = 6 \times 3{,}000{,}000 \times \frac{ba^3}{12} \times \frac{1}{h^2} \times 30 \times 0{\cdot}000006R$$

$$= 270\frac{a^3bR}{h^2} \text{ lb.-in., when all lengths are expressed in inches.}$$

If the load L on the column is equal to $800ab$ lb. and if the extreme fibre stress f_c is 1,200 lb. per sq. in.,

$$\frac{L}{ab} \times \frac{600}{f_c} = 400.$$

From tabulated values, assuming the column is reinforced with 2 per cent. of vertical

steel, this corresponds to a value of e of $0{\cdot}185a$ where e is the eccentricity of application of the load.

Now also $e = \dfrac{M}{L} = \dfrac{270a^3bR}{h^2} \times \dfrac{1}{800ab} = \dfrac{0{\cdot}337a^2R}{h^2}$.

Therefore $0{\cdot}185a = 0{\cdot}337\dfrac{a^2}{h^2}R$, and $R = 0{\cdot}548\left(\dfrac{h}{a}\right)h$.

If $h = 15a$, then $R = 8{\cdot}22h$, and if $h = 10a$, then $R = 5{\cdot}48h$, etc., thus producing the curve shown in *Fig.* 18.12. Referring back to *Fig.* 18.4 and taking an 8-in. wall with $h = 5$ ft., the maximum permissible value of R from *Fig.* 18.12 to keep f_c within 1,200 lb. per sq. in. is $R = 0{\cdot}548 \times \dfrac{5^2}{0{\cdot}667} = 20{\cdot}5$ ft., which confirms that the method of construction will not work when $R = 49{\cdot}5$ ft. If H in *Fig.* 18.4 is increased by sinking the foundations,

$$R = 49{\cdot}5 \text{ ft.} = \frac{0{\cdot}548h^2}{0{\cdot}667} \quad \text{or} \quad h = 7{\cdot}76 \text{ ft.}$$

If the column height in *Fig.* 18.4 is increased from 5 ft. to 7·76 ft. the scheme becomes workable.

The curve shown in *Fig.* 18.12 only holds good for the particular values stated in the figure and makes no allowance for relief of the primary moments due to flexing of the floor slab or rocking over of the foundations. The values are intended only for making and checking preliminary designs. Before starting on a final design the maximum range of temperature appropriate to the particular structure being designed must be decided. The moments (and if necessary, the shears) in the complete structure must be computed and the sections designed accordingly.

Elementary Flat-slab Floors for Tanks

The flat-slab floor shown in *Fig.* 18.13 is without column capitals, drop panels or sunk panels and does not rely for its strength or stiffness on the strength or stiffness of the columns that support it. This is the simplest and oldest form of flat-slab work. The most severe moment occurs over the penultimate column and

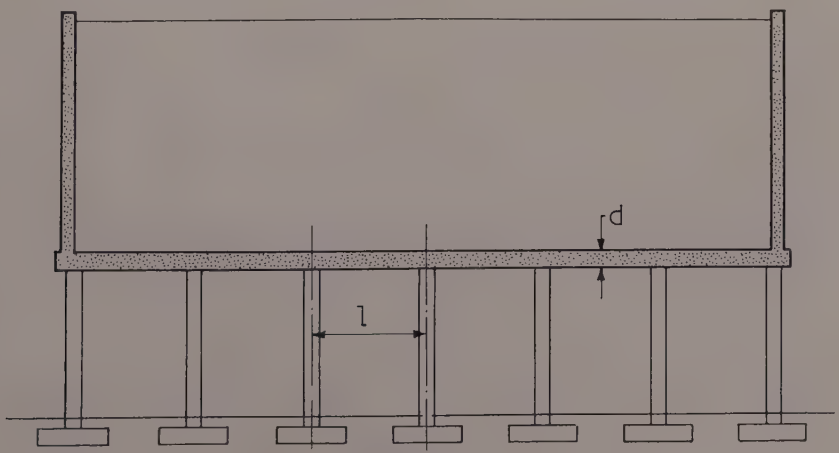


Fig. 18.13.

is $-\frac{1}{16}Wl$ in Strip A or a unit moment of $-\frac{1}{8}W$ per unit width of slab. (The appropriate moments for all types of flat-slab work may be found in the author's 'Reinforced Concrete Design'.)

Since tension occurs on the water face the floor slab must be designed for a resistance moment of about $60bd^2$.

As $W = wl^2$, then $-\frac{1}{8}W = -\frac{1}{8}wl^2$ lb.-ft. per foot width,

$$-\tfrac{1}{8}wl^2 \times 12 = 60 \times 12 \times d^2 \text{ or } w = \frac{480d^2}{l^2} \text{ lb. per sq. ft., where } d \text{ is}$$

the overall thickness of the slab in inches and l is the span in feet.
Thus if $d = 24$ in. and $l = 15$ ft.,

$$w = 480 \times \frac{24^2}{15^2} = 1{,}230 \text{ lb. per sq. ft.}$$

Since a 24-in. slab weighs 300 lb. per sq. ft. this leaves 930 lb. per sq. ft. for the

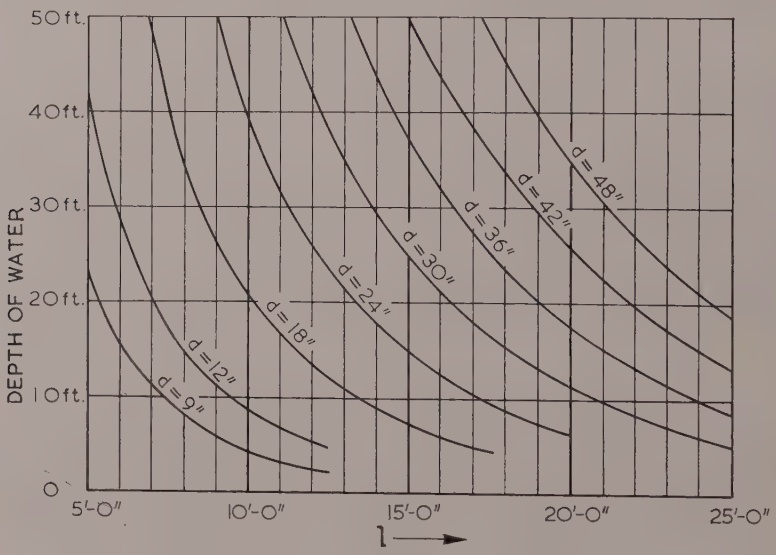


Fig. 18.14.

water load, equivalent to a depth of 14·9 ft. This gives one point on the curve in *Fig.* 18.14, which shows the maximum depth of water carried by various thicknesses of slab over various spans.

Assuming a column reinforced with about 2 per cent. of steel and subjected to an average compression of 800 lb. per sq. in., the column load is wl^2 which equals $480d^2$. With a square column a in. by a in. in size, $800a^2 = 480d^2$ or $a = 0{\cdot}774d$.

The punching shear $= \dfrac{\text{load}}{4ad} = \dfrac{800a^2}{4a \times 1{\cdot}29a} = 155$ lb. per sq. in.

Again, these values are only valid for the exact values of the working stresses that have been assumed. As is clear from *Fig.* 18.5, stresses due to temperature change can completely upset the maximum stresses due to the loading but a reference to *Fig.* 18.12 will at once show if any danger is likely from this source.

Preliminary Design for Circular Tank to Hold 2,000,000 Gallons at 20 ft. above Ground Level

When the dimensions of a tank are not already fixed by site conditions it is generally not possible to deduce what the most economical shape, type or ratio of width-to-depth will be and several alternatives must usually be worked out. 2,000,000 gallons equals 320,000 cu. ft.

Starting with the simplest type, a circular tank with a flat-slab floor similar to *Fig.* 18.13, the tank may be made either

20 ft. deep by 142 ft. 6 in. in diameter,
30 ft. ,, ,, 116 ft. 6 in. ,, ,,
40 ft. ,, ,, 100 ft. 10 in. ,, ,,
or 50 ft. ,, ,, 90 ft. 3 in. ,, ,,.

Starting with the tank 30 ft. deep,

$$\frac{\sqrt{H}}{R} = \frac{\sqrt{30}}{58{\cdot}25} = 0{\cdot}0938.$$

As the floor is not anchored to the ground there will be some stretching and loss of restraint, but this is not a major item in a preliminary design and an approximate value of the wall thickness can be obtained from *Fig.* 3.21. Working to a resistance-moment value of $60bd^2$,

$$\text{if} \quad \frac{\sqrt{H}}{R} = 0{\cdot}0938, \text{ then } T = 0{\cdot}17\frac{H^2}{R} = \frac{0{\cdot}17 \times 30^2}{58{\cdot}25} = 2{\cdot}62 \text{ ft.; say 32 in.}$$

Since the outside panels of the floor will have to be the same thickness as the wall, the simplest solution is to use an elementary flat-slab floor. For a depth of 30 ft. and a floor thickness of 32 in., the columns may be spaced at 15-ft. centres (see *Fig.* 18.14). Assuming the outside columns are under the centre-line of the wall, they will be on a circle 119 ft. 2 in. in diameter. If there is no objection to a central column the diameter may be divided into eight spans of nearly 15 ft. each. The approximate column size is $0{\cdot}774d = 0{\cdot}774 \times 32 =$ say 25 in. by 25 in. The height of the columns h in *Fig.* 18.12 is 20 ft. and $a = 25$ in. $= 2{\cdot}08$ ft.

$$\frac{h}{a} = \frac{20}{2{\cdot}08} = 9{\cdot}6.$$

From *Fig.* 18.12 the maximum allowable value of R/h, to keep f_c within 1,200 lb. per sq. in., is 5·25 or $R = 5{\cdot}25 \times 20 = 105$ ft., against an actual figure of 58·25 ft.

The preliminary design for a 30-ft. depth is shown in the left-hand side of *Fig.* 18.15. The preliminary design for a depth of 40 ft. is shown in the right-hand side of *Fig.* 18.15. The reader is left to calculate and draw preliminary designs for depths of 20 ft. and 50 ft.

In practice it might be necessary to take off quantities of concrete, steel, shuttering, excavation, etc., and make a close estimate of the cost for three or four different schemes including the cost of the roof (if any).

One item that must be considered is the capitalized cost of pumping. If the pressure head in the rising main is ample to reach a height of 70 ft. then the question does not arise, but if all the water has to be pumped from ground level the difference between pumping 40 ft. for a tank 20 ft. deep and 70 ft. for a tank 50 ft. deep might be an important item.

The question as to whether or not it would pay to taper the tank wall is another item that must be investigated before a final design is reached.

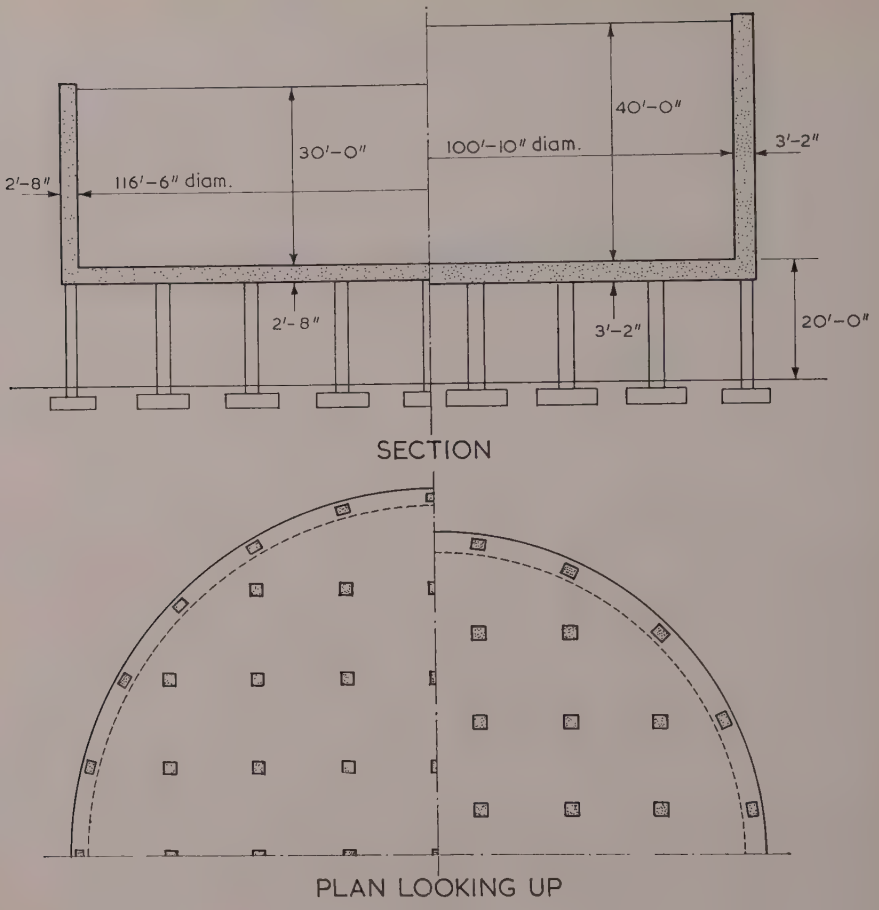


Fig. 18.15.

Some Practical Considerations

Reinforced concrete sleeper walls offer a simple and efficient structural support but they can be an expensive item; they are seldom fully loaded. The walls in *Fig.* 18.4 carry only about 7,500 lb. per foot run. Even if they are only 4 in. in thickness the stress therein is only 157 lb. per sq. in. Leaving large holes to lighten the wall will save a little concrete but introduces complications that cancel any such saving. The shuttering is an expensive item, particularly if the walls are curved. When the slab spans radially the arrangement of bars in radial and circumferential directions is more complicated and more costly than the normal arrangement where all bars are laid in straight lines at right-angles to one another. The design must always include some positive means for supporting the top steel in the slab, such as inclined bars or links. Propping them up on precast concrete blocks is not recommended for watertight work.

The drawings must always show the position of the construction joints and the order in which the work is to be concreted. Special requirements where large quantities of concrete must be placed in a short time must be covered by the specification and by items in the bill of quantities.

CHAPTER NINETEEN

DEEP RECTANGULAR TANKS

DEEP tanks with short sides are reinforced in the horizontal direction, each pair of end walls tying in the sides and resisting bursting under pressure from within.

The analysis which follows also applies to underground tanks and pits subject to external pressure. The bending moments on the walls of such tanks may be found at once from the Theorem of Three Moments. The tank shown in *Fig.* 19.1

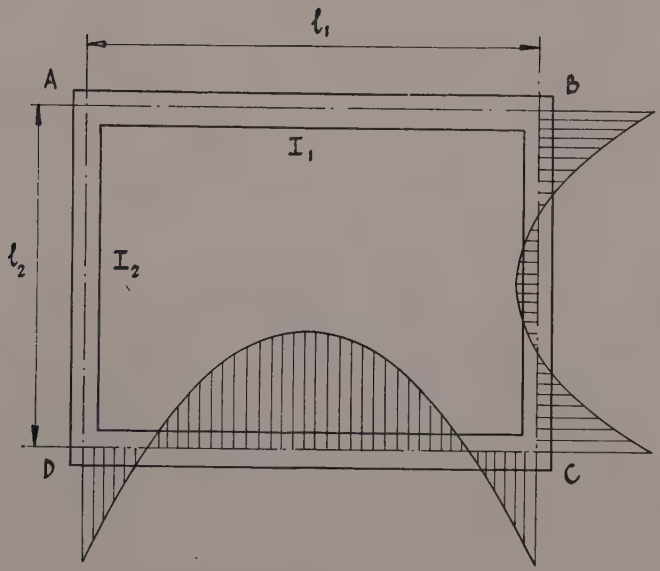


Fig. 19.1.

is l_1 long and l_2 wide, the values of the moments of inertia of a section of the long and short sides being I_1 and I_2 respectively. The tank is subjected to a net unbalanced pressure p. By symmetry, the moments M_A, M_B, M_C, and M_D at all the corners are equal, and

$$M_A = M_B = M_C = M_D = \frac{p\left(\frac{l_1^3}{I_1} + \frac{l_2^3}{I_2}\right)}{12\left(\frac{l_1}{I_1} + \frac{l_2}{I_2}\right)}.$$

If the four walls are of the same thickness, then

$$I_1 = I_2 \quad \text{and} \quad M_A = M_B = M_C = M_D = \tfrac{1}{12}p(l_1^2 - l_1 l_2 + l_2^2).$$

If the pressure is outward these bending moments produce tensile stresses on the inner face of the walls at the corners; if inward the tensile stresses are on the outer face. The diagram of bending moments is shown in *Fig.* 19.1.

In addition to the bending moments the walls have to resist direct tension or

compression depending on whether the net effective pressure acts outwardly or inwardly.

Deep Tanks with Two Compartments

The lengths of the sides of the tank shown in *Fig.* 19.2*a* are l_1, l_2, and l_3. Assuming that the thicknesses are different, then the values of the coefficient of stiffness K (which is the moment of intertia of cross-section I divided by the length l)

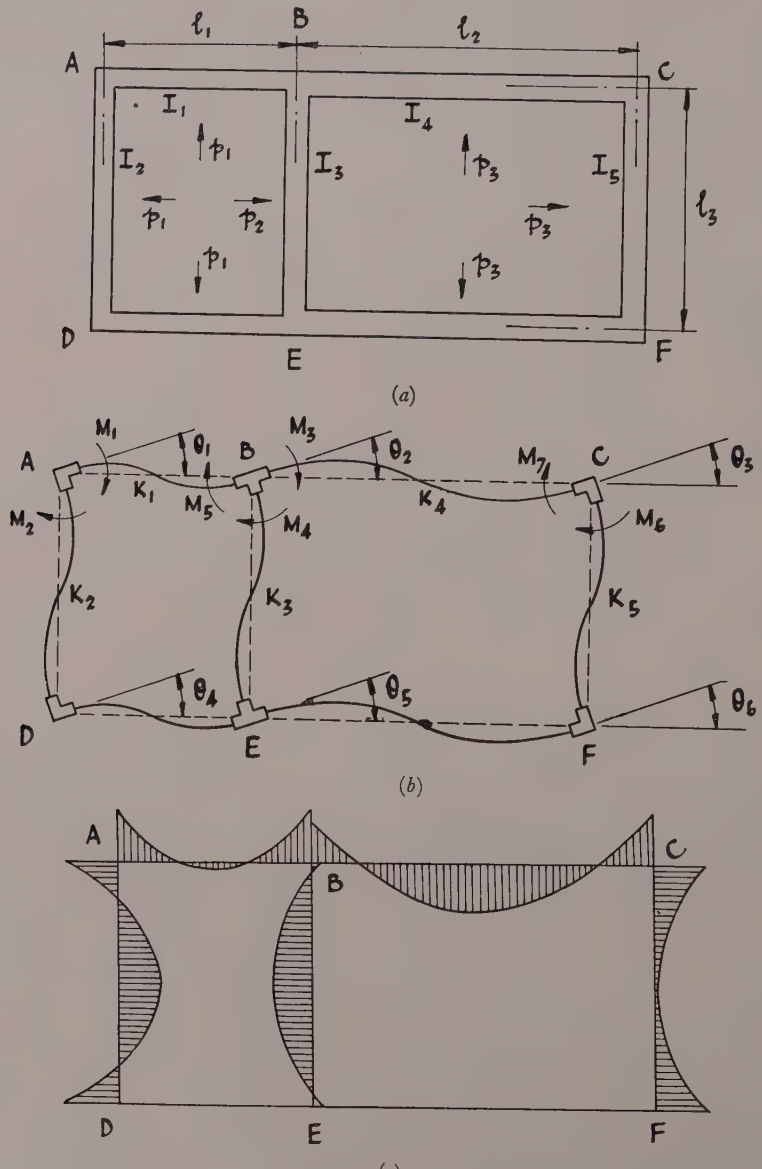


Fig. 19.2.

for the different walls are K_1, K_2, . . . as shown in *Fig.* 19.2*b*. Assume that the compartments are filled to different depths so that the net unbalanced pressures are as follows: on the external walls of the smaller compartment, p_1; on the external walls of the larger compartment, p_3; on the dividing wall, p_2.

The problem may be solved by the displacement method. Drawing the displacements in *Fig.* 19.2*b*, the tabulation is as in the following table.

	M_1	M_2	M_3	M_4	M_5	M_6	M_7
Joints Fixed .	$-p_1l_1^2\times\frac{1}{12}$	$+p_1l_3^2\times\frac{1}{12}$	$-p_3l_2^2\times\frac{1}{12}$	$-p_2l_3^2\times\frac{1}{12}$	$+p_1l_1^2\times\frac{1}{12}$	$-p_3l_3^3\times\frac{1}{12}$	$+p_3l_2^2\times\frac{1}{12}$
Due to θ_1 .	$4EK_1\theta_1$	$4EK_2\theta_1$	—	—	$2EK_1\theta_1$	—	—
,, ,, θ_2 .	$2EK_1\theta_2$	—	$4EK_4\theta_2$	$4EK_3\theta_2$	$4EK_1\theta_2$	—	$2EK_4\theta_2$
,, ,, θ_3 .	—	—	$2EK_4\theta_3$	—	—	$4EK_5\theta_3$	$4EK_4\theta_3$
,, ,, θ_4 .	—	$2EK_2\theta_4$	—	—	—	—	—
,, ,, θ_5 .	—	—	—	$2EK_3\theta_5$	—	—	—
,, ,, θ_6 .	—	—	—	—	—	$2EK_5\theta_6$	—

Due to symmetry, $\theta_4 = -\theta_1$, $\theta_5 = -\theta_2$ and $\theta_6 = -\theta_3$. Due to the equilibrium of the three top corners,

$$M_1 + M_2 = 0 \quad . \quad . \quad . \quad . \quad . \quad (c)$$
$$M_3 + M_4 + M_5 = 0 \quad . \quad . \quad . \quad . \quad . \quad (d)$$
$$M_6 + M_7 = 0 \quad . \quad . \quad . \quad . \quad . \quad (e)$$

Assume that a strip of wall 1 ft. high has 15 ft. of water pressure in the smaller compartment, 10 ft. of water pressure in the larger compartment, and an earth pressure outside all around the tank of 300 lb. per square foot.

$$p_1 = (15 \times 62{\cdot}5) - 300 = 638 \text{ lb. per sq. ft.}$$
$$p_2 = (15 - 10) \times 62{\cdot}5 = 313 \text{ ,, ,, ,, ,,}$$
$$p_3 = (10 \times 62{\cdot}5) - 300 = 325 \text{ ,, ,, ,, ,,}$$

Assume that $l_1 = 8$ ft., $l_2 = 14$ ft. and $l_3 = 10$ ft., that walls AB, AD and DE are 9 in. in thickness, and that walls BC, CF, FE, and BE are 12 in. thick. In units of pounds and feet,

$$K_1 = \frac{1 \times (0{\cdot}75)^3}{12} \times \frac{1}{8} = 0{\cdot}0044. \qquad K_2 = \frac{1 \times (0{\cdot}75)^3}{12} \times \frac{1}{10} = 0{\cdot}00352.$$

$$K_3 = K_5 = \frac{1 \times 1^3}{12} \times \frac{1}{10} = 0{\cdot}00833. \qquad K_4 = \frac{1 \times 1^3}{12} \times \frac{1}{14} = 0{\cdot}00595.$$

Inserting all the numerical values in the foregoing table, the three equations of equilibrium become

$$0{\cdot}02464E\theta_1 + 0{\cdot}0088E\theta_2 = -1{,}914 \quad . \quad . \quad (c)$$
$$0{\cdot}0088E\theta_1 + 0{\cdot}05806E\theta_2 + 0{\cdot}0119E\theta_3 = +4{,}510 \quad . \quad . \quad (d)$$
$$0{\cdot}0119E\theta_2 + 0{\cdot}04046E\theta_3 = -2{,}590 \quad . \quad . \quad (e)$$

Solving these equations: $E\theta_1 = -119{,}100$; $E\theta_2 = +115{,}800$; $E\theta_3 = -98{,}300$. Inserting these three values in the foregoing table (all by slide-rule calculations):

$M_1 = -4{,}480$ lb.-ft. $\quad M_2 = +4{,}482$ lb.-ft. $\quad M_3 = -3{,}712$ lb.-ft.
$M_4 = -678$ lb.-ft. $\quad M_5 = -4{,}393$ lb.-ft. $\quad M_6 = -4{,}350$ lb.-ft.
$M_7 = +4{,}340$ lb.-ft.

from which the complete bending-moment diagram can be easily drawn (*Fig.* 19.2*c*).

To save overlapping at the corners this figure shows the bending-moment

diagrams drawn and hatched on the compression side of the members. The free moment on side BC is $\frac{1}{8}p_3 l_2^2 = \frac{1}{8} \times 325 \times 14^2 = 7{,}960$ lb.-ft. Since the bending moments at the ends of BC are (numerically) 3,712 lb.-ft. and 4,340 lb.-ft., the maximum bending moment near midspan is very nearly 3,390 lb.-ft. (tension at outer face). Since the tank is 9 ft. wide inside there is also a direct tension in the side BC of $\frac{1}{2} \times 9p_3 = 4{\cdot}5 \times 325 = 1{,}460$ lb.

As a matter of interest, $E\theta_1 = -119{,}100$ in pound and foot units. If $E = 3{,}000{,}000 \times 144$ lb. per sq. ft.,

$$\theta_1 = \frac{-119{,}100}{3{,}000{,}000 \times 144} \text{ radians} = -0{\cdot}0158 \text{ deg.};$$

that is, the corner may rotate 5·7 seconds of an arc in a clockwise direction due to this particular loading.

In a rectangular tank above ground level divided by a partition wall into two square compartments, if both compartments are filled to the same depth with liquids of the same densities the moment in the partition wall strip is zero. In the other walls the moments at midspan are $+\frac{1}{24}pl^2$, where l is the length of the side of the square. At all the corners the moments are $-\frac{1}{12}pl^2$.

CHAPTER TWENTY

TANKS BELOW GROUND-WATER LEVEL

The general problem in the design of tanks below ground-water level is indicated in *Fig.* 20.1, the first requirement being that the tank must not float when it is empty. Some inexperienced engineers find it difficult to believe that a large tank can actually float upwards but they may rest assured that this can and does happen.

In some cases it is essential to ensure that there is no leakage of ground water into the tank. In other cases this point is of no importance and the designer may consider relieving the upward pressure on the floor of the tank by inserting relief valves, provided these can be inspected and checked annually. Assuming that this relief cannot be provided, the total weight of the empty tank must exceed the flotation value to give a small factor of safety say 1·10 to 1·25, depending on the accuracy of local records of the highest possible ground-water level.

The total weight will, of course, include all permanent construction such as a roof or superstructure. Sometimes large buildings have basements or elevator pits well below ground-water level that are under the same conditions of flotation, and

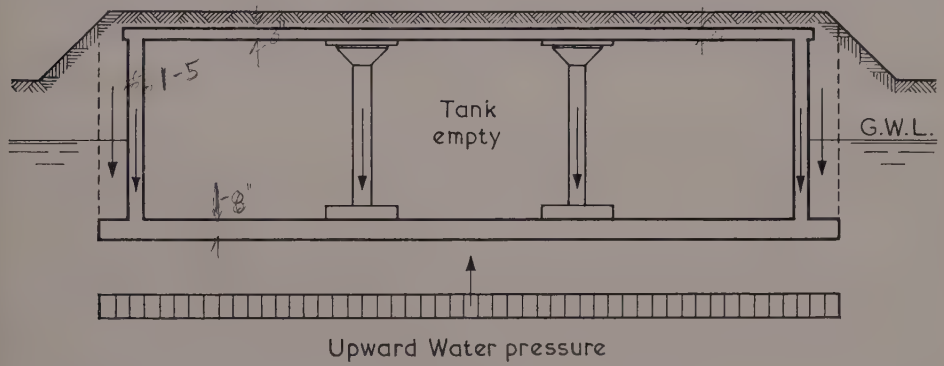


Fig. 20.1.

steps must be taken to prevent their floating during construction before the superstructure is sufficiently complete to hold them down. Sometimes the weight includes a considerable volume of earth. Many British designers would insist that the total weight of permanent construction must be at least equal to the flotation but would include the weight of this earth in the factor of safety.

If the weight of the tank, using the minimum sections necessary to carry the loads, is less than the flotation value the question arises as to the best way of disposing of the additional concrete which is required to weight it down. A cubic foot of concrete above water level weighs 150 lb. but an additional cubic foot below the floor or outside the wall of the tank has an effective weight of only (150 — 62·5) lb. if it is below ground-water level. On a firm site where the floor sits directly on the ground, the addition of a 1-ft. thickness to the roof (if any) increases the moments in both the roof and the floor. Adding a thickness of 1·715 ft. below the floor increases the excavation but does not increase the moments. Increasing the thickness of any member increases the lever arm and decreases the area of steel, but this advantage vanishes when the calculated area of steel falls below 0·3 per cent.

All construction work on ground has a 3-in. layer of mass concrete under it, but this cannot be included in the effective weight as there is no real bond between this layer and the floor proper.

Usually alternative schemes must be prepared and priced.

On soft sites where the tank has to be supported on piles these may be relied on to resist part of the effects of flotation if withdrawal tests show this will be safe.

In exceptional cases the tank may be held down by piles, either driven or bored, which are provided solely for this purpose. The safe resistance to withdrawal of a driven or a bored pile should be established by test loading.

A method of preventing uplift, which has been used in the construction of docks, subjected to upward pressure due to high tides when the dock is empty, is to anchor the bottom slab by screw piles. These generally have a 3-in. to 8-in. solid wrought iron or steel shaft with a cast-iron or cast-steel helicoid having a diameter from 3 to

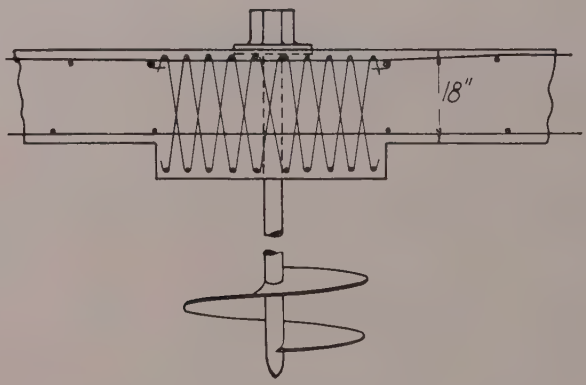


Fig. 20.2.

6 times that of the shaft and a pitch of about one-third of its diameter. The helicoid has seldom more than one turn. The piles are sunk by capstan and winch or by worm gearing.

Where this type of construction is adopted the pile head protrudes through the reinforced floor slab of the dock, and this is strengthened locally to form a pile cap (*Fig.* 20.2). A steel cap is fixed on the head of the pile by a cotter pin passing through the cap and the shaft, and the caps are covered by the plain concrete floor resting on the reinforced slab.

When resisting tension the pull in each screw pile may be considered to be balanced by the dead weight of a truncated cone of earth whose apex is below the blade of the pile. It is a matter of experience in this class of work to determine the probable value of the solid angle at the apex of the cone.

Reinforced concrete screw piles have been used. The reader is referred to descriptions which have appeared elsewhere.

Loading on Walls

If the tank stands in waterlogged granular material the total pressure on the walls under severe earthquake conditions may reach a figure of about 125 lb. per sq. ft. per foot of depth. Pressure from newly-pumped silt or fine sand (or from very soft plastic clay) might also reach a high figure. Arguing from the analogy of

the upward pressure of wet concrete it seems unlikely that the active upward pressure on the floor, even under these conditions, would exceed 62·5 lb. per sq. ft. per foot of depth.

If the lateral pressure of the granular material when dry is 18 lb. per sq. ft. per foot of depth, the specific gravity of the sand grains being 2·65, the total lateral pressure of waterlogged sand under quiescent conditions may be taken as

$$62{\cdot}5 + \left(\frac{1{\cdot}65}{2{\cdot}65} \times 18\right) = 73{\cdot}7 \text{ lb. per sq. ft. per foot of depth.}$$

The author knows no large-scale insitu measurements that would confirm this figure but, if a permissible steel stress of 12,000 lb. per sq. in. is used, no damage is likely if the pressure rises from 73·7 to 100 lb. per sq. ft.

Loading on Floor

On firm but waterlogged sites it is usual to assume that the load is distributed evenly (or in a straight-line variation if eccentrically applied) over the foundation and not concentrated under load-bearing walls or columns. The weight of water standing on the floor and the self-weight of the floor are assumed to pass directly to the foundation. In the tank in *Fig.* 20.1 the effective upward loading on a panel near the centre of the floor, as far as the moments and shears are concerned, is the same as the loading on the panel of roof immediately above plus a small additional item due to the self-weight of the column. This effective loading is the same whether the tank is full or empty and whether the ground water is at its highest or lowest level.

On soft sites where the tank is supported on piles, piers or sleeper walls, the whole of the weight of the tank and contents less the uplift from ground water at its minimum level (natural or pumped-down) must be carried to the points of support.

Design of Walls

It will be clear from Chapters 9 and 11 that the design of reinforced concrete cantilevered walls becomes impracticable when upward water pressure develops on the underside of the base although gravity walls of mass concrete may be used for small depths. The advantages of a circular wall are obvious and for a large tank this is often the only reasonable solution. The critical condition of a ring carrying a uniform external pressure p is given by the formula $p = 3EI/R^3$.

If $p = 62{\cdot}5h$ and $E = 3{,}000{,}000 \times 144$ lb. per sq. ft., the minimum buckling thickness T of a free-standing wall is given by $T = \sqrt[3]{h} \times \frac{R}{120}$.

If $R = 100$ ft. and $h = 40$ ft., then $T = 2{\cdot}85$ ft. This formula could only apply to a wall with a sliding joint at the base surrounded by water. In *Fig.* 20.3, the critical values of T for depths up to 50 ft. for a tank 200 ft. in diameter are shown. Since T varies directly as R the thicknesses for a tank 100 ft. in diameter ($R = 50$) would be one-half of those shown in *Fig.* 20.3.

Very heavy work which falls within the province of dock and harbour engineering is outside the scope of this book.

For a deep tank with short plan dimensions the walls may span horizontally. The moments may be found by the methods described in Chapter 19. For a long narrow tank any of the sections shown in *Fig.* 20.4*a*, *b* or *c* could be used. The

design in *Fig.* 20.4*b* can be adopted only if the internal counterforts and floor beams do not interfere with the use of the tank. The design in *Fig.* 20.4*c*, with a single tie above water level, may be used for widths of up to 50 ft. if the bottom slab is heavy enough to resist the upward water pressure. Beyond this

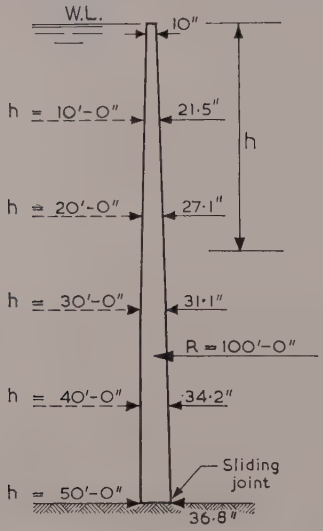


Fig. 20.3.

point the thermal expansion and contraction of the tie poses a problem (see Chapter 12). A second strut halfway down, if permitted, would give a cheaper design. The design in *Fig.* 20.5 is seldom used owing to the difficulties of construction. A deep narrow beam in waterlogged ground may be built inside a close-timbered trench, the timbering, being left in, but if the soil is fairly stiff it is better to excavate

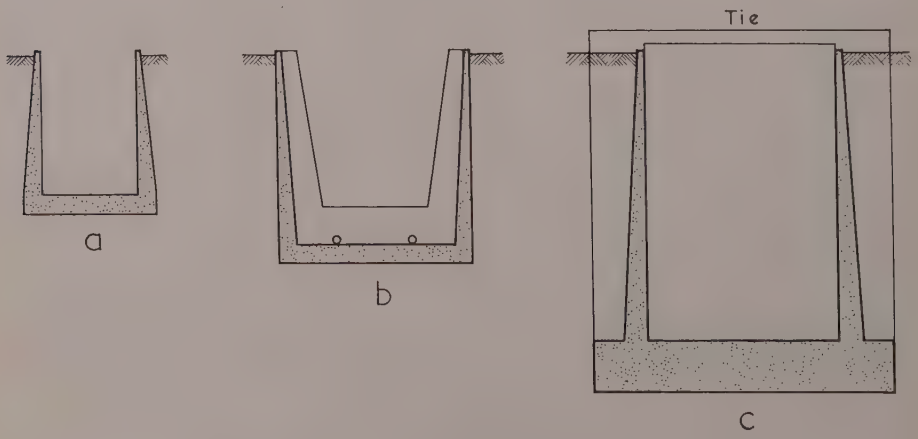


Fig. 20.4.

the ground to its natural slope, as in *Fig.* 20.5, and form a trench in mass concrete inside which the beam can be cast.

A preliminary design for one internal bay of a large drainage reservoir is shown in *Fig.* 20.6. The reservoir sits on water-bearing ballast. Under the worst winter

conditions the ground water may rise to the level shown. Under summer conditions it may fall well below the underside of the floor. The reservoir has to be 15 ft. deep and in order to get a closer look at the problems involved, a provisional design may be prepared assuming that the spacing of the columns is equal to the

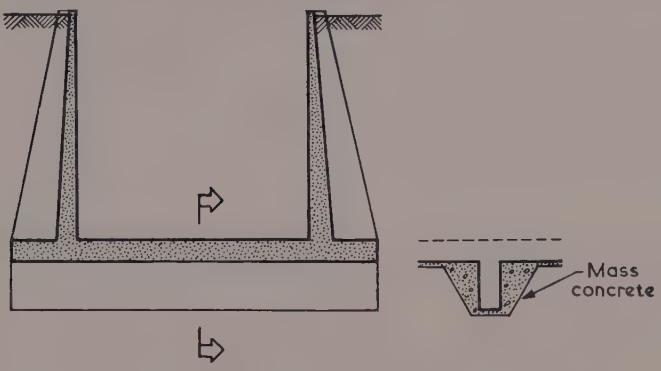


Fig. 20.5.

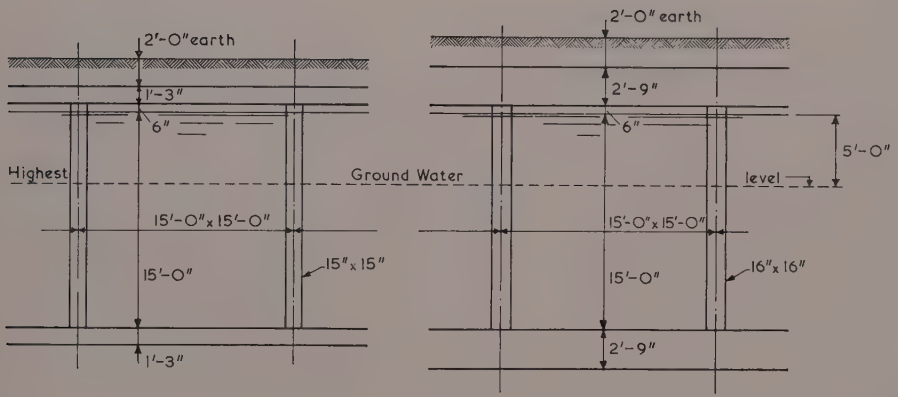


Fig. 20.6.

depth of water. Using an elementary flat-slab design, making the thickness of the floor and roof equal to one-twelfth of the span and guessing the sizes of the columns, the dimensions in the left-hand side of the figure are obtained.

The load on the roof is 2 ft. of earth plus some imposed load—say a total of 250 lb. per sq. ft., and the self-weight of the slab is 180 lb. per sq. ft. Thus the total load on the panel is $15 \times 15\ (250 + 180) = 96{,}900$ lb.

Taking controlling unit moments of $-\frac{1}{8}W$ and $+\frac{1}{10}W$ for end bay conditions:

$$-\tfrac{1}{8}W = -\frac{96{,}900 \times 12}{8} = -145{,}000 \text{ lb.-in. per foot.}$$

$$+\tfrac{1}{10}W = +\frac{96{,}900 \times 12}{10} = +116{,}000 \text{ lb.-in. per foot.}$$

The effective upward load on the floor will be very nearly the same as the load on the roof. Taking the view that the underside of the floor is a face ' in contact with water ' and designing the floor to carry —145,000 lb.-in. (assuming a resistance

moment of $60bd^2$) a thickness of 14·2 in. is required. This design is sound structurally but the possibility of flotation must now be checked.
Weight of one bay (ignoring earth):

Roof:	$15 \times 15 \times 180$	= 40,500 lb.
Column:	$15{\cdot}5 \times 225$	= 3,500 lb.
Floor:	$15 \times 15 \times 180$	= 40,500 lb.
		84,500 lb.

Flotation: $15 \times 15 \times 11{\cdot}25 \times 62{\cdot}5 = 158{,}000$ lb.
Uplift = 158,000 − 84,500 = 73,500 lb.

$$\frac{73{,}500}{15 \times 15} = 327 \text{ lb. per sq. ft.}$$

If the roof and floor are to be made the same thickness and both are increased in thickness by 1 ft.:

Addition of 1 ft. to roof =	144 lb. per sq. ft.
Addition of 1 ft. to floor =	81·5 lb. per sq. ft.
	225·5 lb. per sq. ft.

$$\frac{327}{225{\cdot}5} = 1{\cdot}45 \text{ ft., say 18 in.}$$

If the thicknesses of both floor and roof are made 15 + 18 = 33 in., the bay will be just safe when empty with no earth on the roof. Earth = say $15 \times 15 \times 200$ lb. = 45,000 lb. With the earth in place the design shown on the right-hand side of *Fig.* 20.6 has a factor of safety against flotation of 1·27.

The effective upward load on the floor including the earth and imposed load is now 663 lb. per sq. ft. and $W = 663 \times 15 \times 15 = 149{,}000$ lb.
Controlling $M = \frac{1}{8} \times 149{,}000 \times 12 = 224{,}000$ lb.-in. per foot.

$$\text{At 12,000 lb. per sq. in., } A_{st} = \frac{224{,}000}{12{,}000 \times 0{\cdot}9 \times 31} = 0{\cdot}67 \text{ sq. in.}$$

$$0{\cdot}3 \text{ per cent. of the section } = \frac{0{\cdot}3}{100} \times 12 \times 33 = 1{\cdot}19 \text{ sq. in.}$$

This is 78 per cent. more steel than is required to resist the maximum moment. If the designer is not prepared to provide less than 0·3 per cent. of steel where the maximum moments occur, the moment can be increased by 78 per cent., which means increasing W by 78 per cent. or increasing the column spacing from 15 ft. by 15 ft. to 20 ft. by 20 ft. Any column spacing up to 20 ft. that fits in with the overall width of the reservoir could therefore be used if 0·3 per cent. of steel is provided to resist the main moments. A slab 33 in. thick with a resistance moment of $60bd^2$ will resist 785,000 lb.-in., which is 3·5 times 224,000 lb.-in. The column spacing could then be increased to 28 ft. by 28 ft. without overstressing the concrete in tension, but A_{st} would have to be increased from 1·19 sq. in. to 2·55 sq. in.

This is as far as a preliminary design can go. From this point it becomes necessary to make a close study of site conditions before making final adjustments.

A preliminary design for a tank 15 ft. by 15 ft. by 10 ft. deep is shown in *Fig.* 20.7. Under flood conditions the ground water may rise to ground level. The tank is to have a factor of safety against flotation of 1·1.

Design may be commenced assuming that the walls and floor are the same

thickness. This thickness can then either be calculated or determined by trial and error.

If the thickness of the walls and floor is 2 ft. and it is assumed that the concrete weighs 144 lb. per cu. ft.:

$$\text{Weight} = 144[(19^2 \times 12{\cdot}5) - (15^2 \times 10{\cdot}5)] = 311{,}000 \text{ lb.}$$

$$\text{Flotation} = 62{\cdot}5 \times 19^2 \times 12 = 271{,}000 \text{ lb.}$$

$$\text{Factor of safety} = \frac{311{,}000}{271{,}000} = 1{\cdot}15.$$

Assuming the weight is spread evenly over the foundation,

Upthrust =	862 lb. per sq. ft.
The floor weighs $311{,}000/19^2$ =	288 lb. per sq. ft.
	574 lb. per sq. ft.

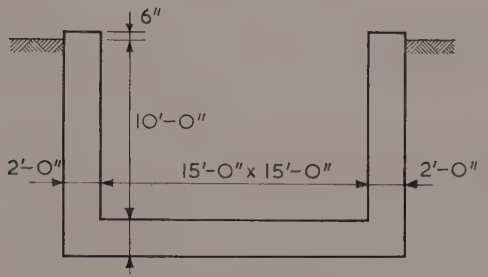


Fig. 20.7.

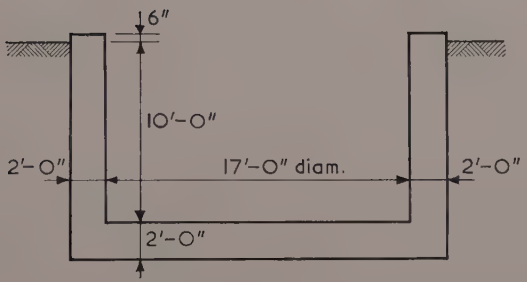


Fig. 20.8.

The maximum ground-water pressure under the floor is

$$12 \times 62{\cdot}5 = 750 \text{ lb. per sq. ft.}$$

Designing the floor as a 17-ft. by 17-ft. mesh panel, $W = 17^2 \times 574 = 166{,}000$ lb.
Designing the walls to cantilever from the floor,

$$\text{Water pressure} = 0{\cdot}5 \times 62{\cdot}5 \times 10^2 = 3{,}130 \text{ lb. per foot run.}$$

$$M = 3{,}130 \times 40 = 125{,}000 \text{ lb.-in.}$$

If the tank is built in open excavation under dry summer conditions the edge of the floor slab will have this moment imposed on the primary moment of $\frac{1}{24}W$.
Unit M in floor $= (\frac{1}{24} \times 166{,}000 \times 12) + 125{,}000 = 208{,}000$ lb.-in.

$$A_{st} \text{ at base of wall} = \frac{125{,}000}{12{,}000 \times 0{\cdot}86 \times 22} = 0{\cdot}55 \text{ sq. in.}$$

$$A_{st} \text{ in floor} = \frac{208{,}000}{12{,}000 \times 0{\cdot}86 \times 21{\cdot}5} = 0{\cdot}94 \text{ sq. in.}$$

If $\frac{3}{4}$-in. diam. bars are provided at 9-in. centres in each wall and these are run across the floor, this arrangement gives 0·59 sq. in. in the wall and 1·18 sq. in. near the top of the floor (see the left-hand side of *Fig.* 20.9). This leaves an area of

$$(1{\cdot}18 - 0{\cdot}94) = 0{\cdot}24 \text{ sq. in.}$$

to take the direct tension if necessary. In most cases of tanks sitting directly on the ground (as distinct from those supported on piles) this may be assumed to be taken by friction or counterpressure. If friction and counterpressure are ignored,

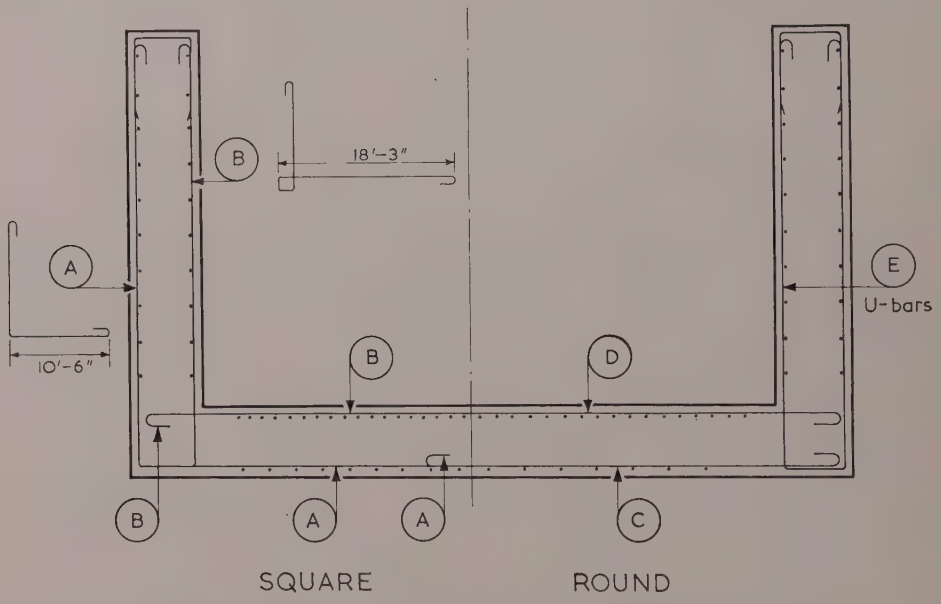


Fig. 20.9.

and all the tension is taken on the top steel, the additional area required would be $\frac{3{,}130}{12{,}000} = 0{\cdot}261$ sq. in. so this requirement is practically covered in any case.

With such thick sections in a small tank the requirement of a minimum steel area of 0·15 per cent. of *bd* is often the deciding factor.

0·15 per cent. of 12 in. × 24 in. = 0·432 sq. in.;

say not less than $\frac{3}{4}$-in. diam. bars at 12-in. centres.

In *Fig.* 20.8 an alternative arrangement to *Fig.* 20.7 is shown, using a circular tank of similar capacity.

Calculating the restraint moment and shear, $\frac{H}{\sqrt{TR}} = \frac{10}{\sqrt{2 \times 8{\cdot}5}} = 2{\cdot}43$.

From *Fig.* 3.13, $M = 0{\cdot}2 \times 62{\cdot}5 \times 10 \times 2 \times 8{\cdot}5 = 2{,}120$ lb.-ft.
$= 25{,}500$ lb.-in.

From *Fig.* 3.11, $F = 0{\cdot}26 \times 62{\cdot}5 \times 10^2 = 1{,}620$ lb.

The net upward load on the floor is 518 lb. per sq. ft.

Designing the floor as a mesh panel 19 ft. in diameter and adding the edge moment,
Unit $M = (\frac{1}{24} \times 518 \times 19^2 \times 12) + 25{,}500 = 119{,}100$ lb.-in.

$$A_{st} \text{ for } M = \frac{119{,}100}{12{,}000 \times 0{\cdot}86 \times 21{\cdot}5} = 0{\cdot}538 \text{ sq. in.}$$

$$\text{Direct tension} = \frac{1{,}620}{12{,}000} = 0{\cdot}135 \text{ sq. in.}$$

$$0{\cdot}673 \text{ sq. in.};$$

say $\frac{5}{8}$-in. diam. bars at $5\frac{1}{2}$-in. centres (0·67 sq. in.).
The restraint moment and maximum hoop tension both require less than 0·15 per cent. of steel.

Preliminary steel details for both square and round tanks are shown in *Fig.* 20.9.

Tanks as Caissons

Tanks below ground level may be sunk as caissons. For further information see the Author's ' Design and Construction of Foundations '.

Earth-loaded Tanks

In *Fig.* 20.10 a preliminary design is shown for an open-topped tank 30 ft. by 30 ft. by 10 ft. deep, for which a substantial part of the uplift is taken by earth filling. The requirements are that the walls must resist pressure from 10 ft. of water before the earth backing is placed and that the tank must not float when ground-water level reaches the top of the wall.
M from 10-ft. head of water $= 0{\cdot}167 \times 62{\cdot}5 \times 10^3 \times 12 = 125{,}000$ lb.-in.
d required for resistance-moment value of $60bd^2 = 13{\cdot}2$ in.; say 14 in.
Guessing that the floor will be a little thicker than the wall, a thickness of 18 in.

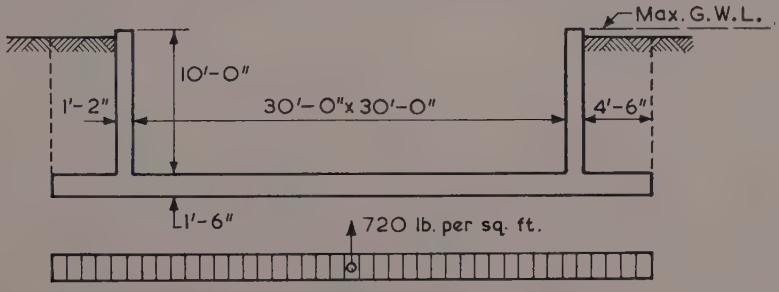


Fig. 20.10.

may be assumed. Ignoring the projection of the floor slab and dealing with the stability of the tank when this is omitted:

Weight of walls: $(60 + 64{\cdot}67) \times 10 \times 168 = 209{,}500$ lb.
Net floor: $32{\cdot}33^2 \times 216 = 226{,}000$ lb.
$435{,}500$ lb.

The flotation would be $62{\cdot}5 \times 32{\cdot}33^2 \times 11{\cdot}5$ ft. $= 753{,}000$ lb.
With absolutely no outside projection there will be a net uplift of (753,00 − 435,500) or 317,500 lb. Every square foot of projection will carry 6 in. of water above ground level, 9 ft. 6 in. of earth and the self-weight of the 18-in. floor, a total of say

$(31{\cdot}25 + 950 + 216) = 1{,}197$ lb.; its displacement is $1{\cdot}0 \times 1{\cdot}0 \times 11{\cdot}5$ cu. ft., equal to $11{\cdot}5 \times 62{\cdot}5 = 720$ lb. Every square foot of projection thus gives $(1{,}197 - 720)$ or 477 lb. of additional load. To balance 317,500 lb., an area of $\frac{317{,}500}{477} = 663$ sq. ft. is required. The outside dimension will then be $\sqrt{663 + 32{\cdot}33^2} = 41{\cdot}35$ ft., which means that a 4-ft. 6-in. projection is required all round. Checking this:

Weight of walls		=	209,500 lb.
18-in. floor	$= 41{\cdot}33^2 \times 216$	=	370,000 lb.
earth	$= 663 \times 9{\cdot}5 \times 100$	=	630,000 lb.
6-in. water	$= 663 \times 31{\cdot}25$	=	20,700 lb.
			1,230,200 lb.
Flotation	$= 41{\cdot}33^2 \times 11{\cdot}5 \times 62{\cdot}5$	=	1,230,000 lb.

Assuming that the total weight is spread uniformly over the base.

$$\text{Pressure} = \frac{1{,}230{,}000}{41{\cdot}33^2} = 720 \text{ lb. per sq. ft.}$$

The net uplift under the centre of the tank is $(720 - 216) = 504$ lb. per sq. ft. The net downward load on the projection (as already calculated) is 477 lb. per sq. ft. Moment at base of wall from ground water and submerged earth:

$$0{\cdot}167 \times 62{\cdot}5 \times 10^3 \times 12 = 125{,}000 \text{ lb.-in.}$$
$$0{\cdot}167 \times 11{\cdot}2 \times 9{\cdot}5^3 \times 12 = 19{,}000 \text{ lb.-in.}$$
$$144{,}200 \text{ lb.-in.}$$

Cantilever moment on toe $= 0{\cdot}5 \times 477 \times 4{\cdot}5^2 \times 12$ or 58,000 lb.-in.
Designing the main part of the floor as a mesh panel 31 ft. 2 in. by 31 ft. 2 in., and carrying an upward net pressure of 504 lb. per sq. ft., it will be seen from *Fig.* 20.11 (which shows the joint at the base of the wall) that this panel has a reversed

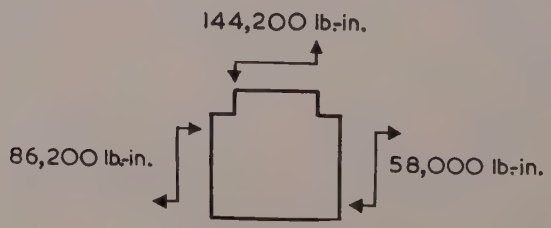


Fig. 20.11.

moment of $(144{,}200 - 58{,}000)$ or 86,200 lb.-in. per foot run of wall along its perimeter.
Average positive moment in panel

$$= \tfrac{1}{24}W - 86{,}200 = \tfrac{1}{24}(504 \times 31{\cdot}17^2) \times 12 - 86{,}200 = 158{,}800 \text{ lb.-in.}$$

This figure assumes that absolutely uniform conditions occur round the whole tank and is a minimum value which occurs under the best site conditions. The author would generally not go below $\frac{1}{30}W$ for an average site or

$$\tfrac{1}{30}(504 \times 31{\cdot}17^2) \times 12 = 196{,}600 \text{ lb.-in.}$$

With an 18-in. slab this only represents a moment of $50bd^2$.
Only the weight of the earth standing directly over the floor has been included.

This is equivalent to assuming that, if the tank floated, the earth would rupture on the vertical planes shown by broken lines in *Fig.* 20.10. Most soils would tend to rupture on an inclined plane, thus tending to increase the effective downward load from the earth but this increase might be small in waterlogged non-cohesive ground. Ignoring this additional safeguard the factor of safety against uplift is 1·00. If the concrete weighs less than 144 lb. per cu. ft. or the earth less than 100 lb. per cu. ft., or if the ground water turns out to be saline and weighs 64 lb. per cu. ft., the factor may be nearer 0·95. In view of the little extra material required, an increase in the width of the toe from 4 ft. 6 in. to 5 ft. is indicated.

CHAPTER TWENTY-ONE

PYRAMIDAL AND CONICAL TANKS

Deep pyramidal settling-tanks were once fairly common. Today a shallow tank with only a small slope on the floor and provided with sweeping apparatus is more usual. Deep tanks are mostly, if not wholly, below ground and their design depends on the type of soil.

Tanks in Hard Dry Soil

In hard dry ground such as solid chalk the excavation may be trimmed to shape and will stand without support, the sloping slab being cast against the soil. A nominal 6-in. slab with 0·3 per cent. of steel in both directions, as shown in

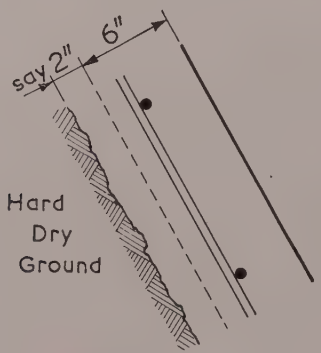


Fig. 21.1.

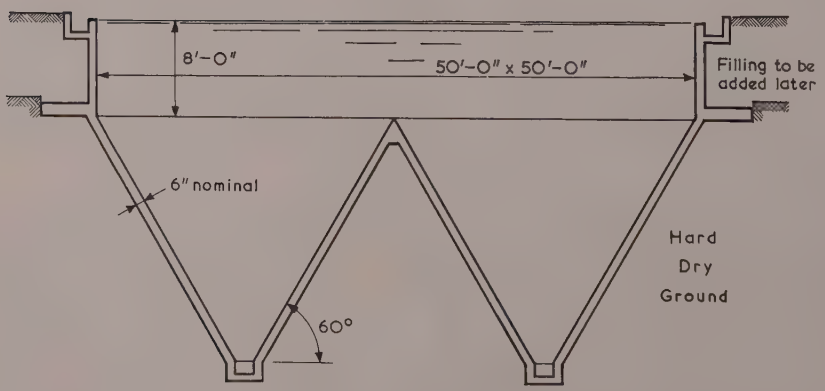


Fig. 21.2.

Fig. 21.1, is ample for strength, but it is necessary to allow another 2 in. or so to make sure of maintaining the correct concrete cover to the outside face of the steel if small pieces of soil break off during concreting.

In rock a 3-in. facing of dense mass concrete may be sufficient, but there is sure

to be overbreak and a ' 3-in. slab ' may have an average thickness of 9 in. or more. In *Fig.* 21.2 a typical section through a group of four pyramidal tanks 25 ft. by 25 ft. in size, constructed in hard dry ground, is shown.

The beginner should notice that the slope down the diagonal of a square tank is less than the slope down the side. If the slope on the diagonal is 45 deg., the side slopes at 54 deg. 45 min. If the side slopes at 60 deg., the slope down the diagonal is 50 deg. 47 min.

The only difficulty in design arises if part of the tank lies above the original ground level, as in *Fig.* 21.2, and shown in detail in *Fig.* 21.3. The water pressure of 2,050 lb. per foot run of wall is resisted by three forces R_1, R_2 and R_3. The wall, which cantilevers from the base, is assumed to rotate round its inside edge producing a triangular reaction R_2. The value of this reaction is obtained by taking moments about the point where R_1 and R_3 intersect. The value of R_3 may be obtained by

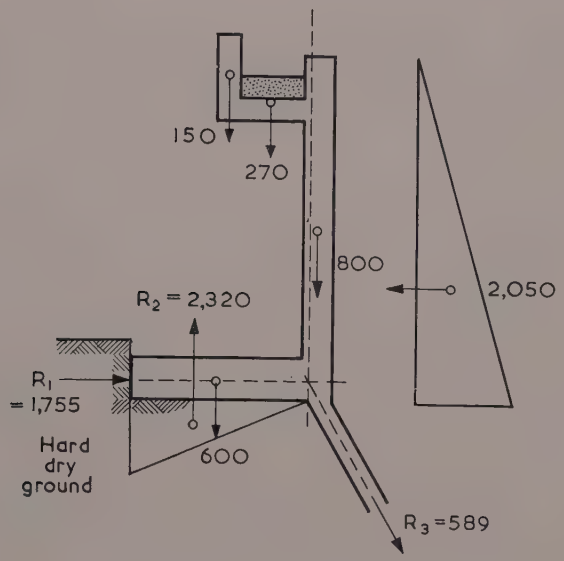


Fig. 21.3.

resolving vertically and R_1 may then be found by resolving horizontally. External earth pressure, when the tank is empty, tends to drive the wall into the hole and there may not be sufficient friction between the underside of the base and the soil to keep it in place. The pressure from 8 ft. of filling will not exceed 640 lb. per foot run of wall. If the base of the wall is 12 in. by 56 in. in size and is designed as a square collar beam running round the four tanks, the beam spans 54 ft. 8 in. If this base carries all the earth pressure, $M = \frac{1}{12} \times 640 \times 54{\cdot}67^2 \times 12 = 1{,}740{,}000$ lb.-in. With a resistance-moment value of $60bd^2$, the safe moment on the base acting as a beam is $60 \times 12 \times 56^2 = 2{,}260{,}000$ lb.-in.

In designing the section at the base of the vertical wall the moment from the eccentric loads from the overflow trough must be added to the cantilever moment from the water pressure.

The design of the upper part of an isolated conical tank in hard dry ground is similar to the design shown in *Figs.* 21.9, 21.10, and 21.11.

Tanks in Soft Wet Ground

If a tank is to be constructed in soft wet ground the bottom of the tank must be taken down to a hard firm foundation. In *Fig.* 21.4 theoretical thin-walled tanks in very soft ground which is unable to supply any vertical or horizontal support are shown, the whole weight of the tank and contents being carried on the bottom point. As far as internal pressures are concerned the tanks are in the same state as tanks built wholly above ground level.

The sloping walls of the square tank may be designed as spanning a distance L under a pressure p. As span L increases, p decreases. The moment of $\frac{1}{12}pL^2$ is

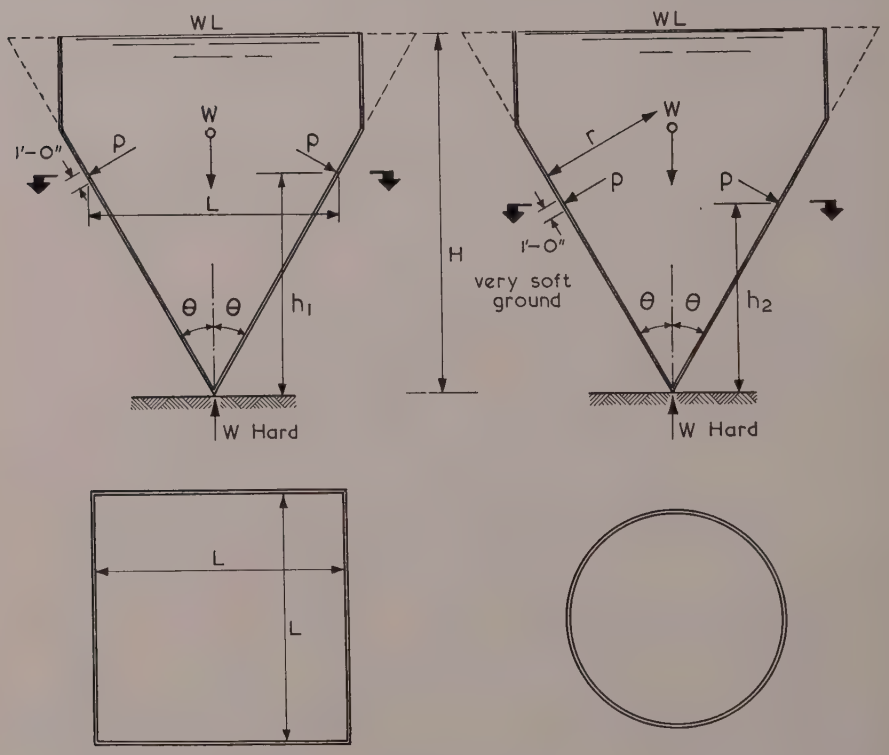


Fig. 21.4.

a maximum when $h_1 = \frac{2}{3}H$ and $M = \frac{4}{81}wH^3 \tan^2\theta$, where w is the weight per cu. ft. of the contents. If $H = 25$ ft., $\theta = 30$ deg. and $w = 64$ lb. per cu. ft.,

$$M = \tfrac{4}{81} \times 64 \times 25^3 \times \tfrac{1}{3} = 16{,}500 \text{ lb.-ft. per foot width.}$$

If this has to be resisted with a resistance-moment value of $60bd^2$, $d = 16{\cdot}6$ in. In practice the component of the self-weight at right-angles to the slope must be added to p. If the thickness of the sloping wall is 18 in. it weighs 216 lb. per sq. ft., and if $\theta = 30$ deg., the component is $216 \sin\theta = 108$ lb. per sq. ft.; if $L = 19{\cdot}25$ ft., the moment is increased by $\frac{1}{12} \times 108 \times 19{\cdot}25^2 = 3{,}340$ lb.-ft.

The sloping wall of the conical tank may be designed as a slab subjected to ring tension. The tension per foot height of slope in *Fig.* 21.4 is pr, where r is the radius of curvature measured at right-angles to the slope (not the radius in a horizontal plane). As h_2 increases, r increases but p decreases. Ignoring the self-weight of

the wall, the ring tension is a maximum when $h_2 = 0{\cdot}5H$. If $H = 25$ ft. and $\theta = 30$ deg., then $r = 8{\cdot}33$ ft. when $h_2 = 12{\cdot}5$ ft.

Assuming $w = 64$ lb. per cu. ft., $p = 64 \times 12{\cdot}5 = 800$ lb. per sq. ft. and ring tension $= 800 \times 8{\cdot}33 = 6{,}650$ lb. per foot. If the wall is 8 in. in thickness the ring tension in the concrete is only $\dfrac{6{,}650}{8 \times 12} = 69$ lb. per sq. in.

The component of the self-weight of an 8-in. wall is 50 lb. per sq. ft. and this would increase the ring tension by 6·25 per cent.

Flotation

Although there may be a considerable amount of mass concrete outside the tank, experience shows that there is no adhesion between weak mass concrete and reinforced concrete cast against it, either some days or weeks later.

The reinforced concrete structure must be sufficiently heavy to resist flotation on its own without any assistance from the mass concrete outside it. To obtain some approximate figures consider the thin theoretical pyramidal tank in *Fig.* 21.5.

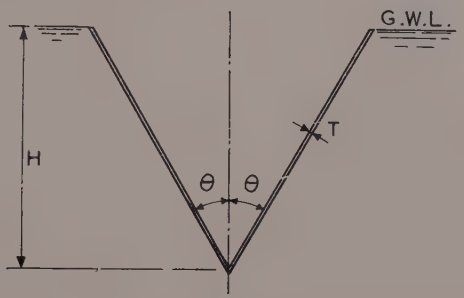


Fig. 21.5.

The figure shows the portion of the tank below ground-water level. Assuming the walls are of uniform thickness of T feet and the material of which they are made weighs w_c lb. per cu. ft. Then

$$\text{weight} = \frac{4H^2 \tan\theta \; Tw_c}{\cos\theta}.$$

If w is the weight of the ground-water per cu. ft., the flotation is $\dfrac{4H^3 \tan^2\theta w}{3}$.

If the tank is just in equilibrium,

$$\frac{4H^2 \tan\theta \, Tw_c}{\cos\theta} = \frac{4H^3 \tan^2\theta w}{3}, \quad \text{and} \quad T = \frac{H \sin\theta}{3} \times \frac{w}{w_c}.$$

This relation is also true for a conical tank. If $\theta = 30$ deg., $w = 62{\cdot}5$ and $w_c = 150$ lb. per cu. ft., $T = 0{\cdot}0695H$. If $H = 20$ ft., then $T = 1{\cdot}39$ ft.

Every practical case is complicated by the actual thickness of the wall (assumed as negligible in *Fig.* 21.5), by the weight of those parts of the walls and overflow channels above ground-water level, and by various local thickenings.

When the tank is empty the walls must resist external pressure from the ground water, but in this case the component of their self-weight decreases the effective pressure and the thickness required to resist ground-water pressure will be less

than the thickness required to hold the internal pressure. In *Fig.* 21.6 a preliminary design is shown for a set of four tanks having the same internal dimensions as those in *Fig.* 21.2. The intersection point of the internal slopes lies 21·65 ft. below the foot of the vertical wall, corresponding to say, $H = 30$ ft. in *Fig.* 21.4. The maximum

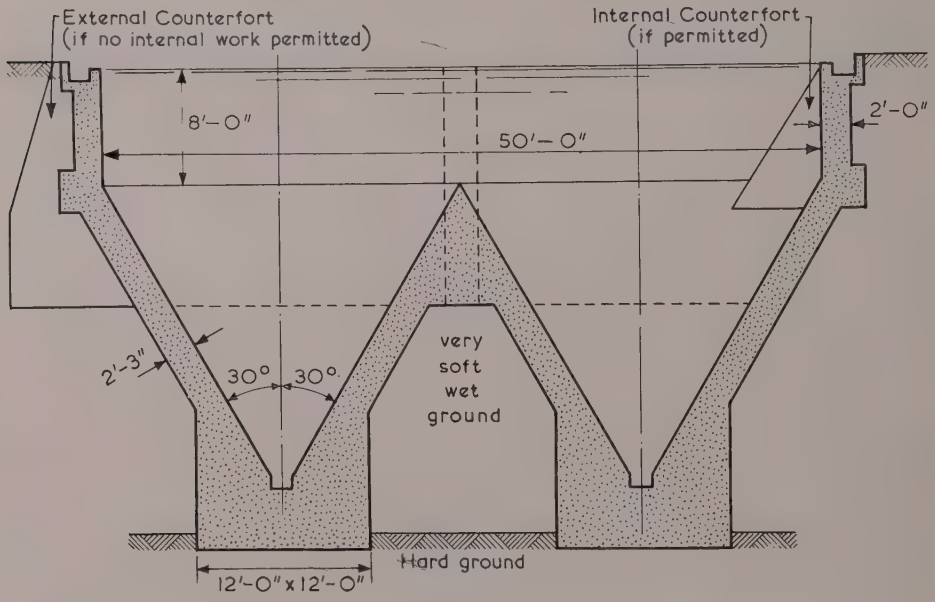


Fig. 21.6.

moment in the sloping wall occurs about $0{\cdot}33H$ or 10 ft. down, where $p = 10 \times 64$ or 640 lb. per sq. ft. The effective span L at this point, measured centre-to-centre of walls, is about 25 ft. Assuming the thickness of the wall is 2 ft., the resolved component is 150 lb. per sq. ft.

$$M = (640 + 150) \times 25^2 \times \frac{12}{12} = 494{,}000 \text{ lb.-in.}$$

With a resistance-moment value of $60bd^2$, $d = 26{\cdot}2$ in.; say 27 in.

There is also a direct tension of about 9,000 lb. and the final design should be adjusted to cover this. The intersection point of the outer slopes lies 54 in. below the intersection point of the inner slopes or, says, 34 ft. 6 in. below water level. Working backward from the flotation formula relating to *Fig.* 21.5, $T = 0{\cdot}0695H$. If $T = 2{\cdot}25$ ft., then $H = 32{\cdot}4$ ft. Approximately speaking the tank would be on the point of floating if the ground-water level rose to within 2 ft. of the top of the vertical wall. This must be checked exactly when the final design is prepared. The weight of the contents of one compartment is

$$64[(25 \times 25 \times 8) + \tfrac{1}{3}(25 \times 25 \times 21{\cdot}65)] = 610{,}000 \text{ lb.}$$

The approximate weight of the wall is

$$\frac{4H^2 \tan\theta T w_c}{\cos\theta} = \frac{4 \times 34{\cdot}5^2 \times 0{\cdot}577 \times 2{\cdot}25 \times 150}{0{\cdot}866} = 1{,}070{,}000 \text{ lb.}$$

The approximate total is therefore 1,680,000 lb. or 750 tons on the foundation, which is shown as 12 ft. square, giving a bearing pressure of $5\frac{1}{4}$ tons per sq. ft. Assuming that all internal ties and internal counterforts are forbidden, the internal pressure on the vertical wall can be spanned horizontally and picked up by U-shaped external counterforts, as shown on the left-hand side of *Fig.* 21.6 and in *Fig.* 21.7. The horizontal leg of this counterfort consists of a beam 6 ft. 6 in. deep and 27·7 in. wide at the top, surmounted by a precast coping.

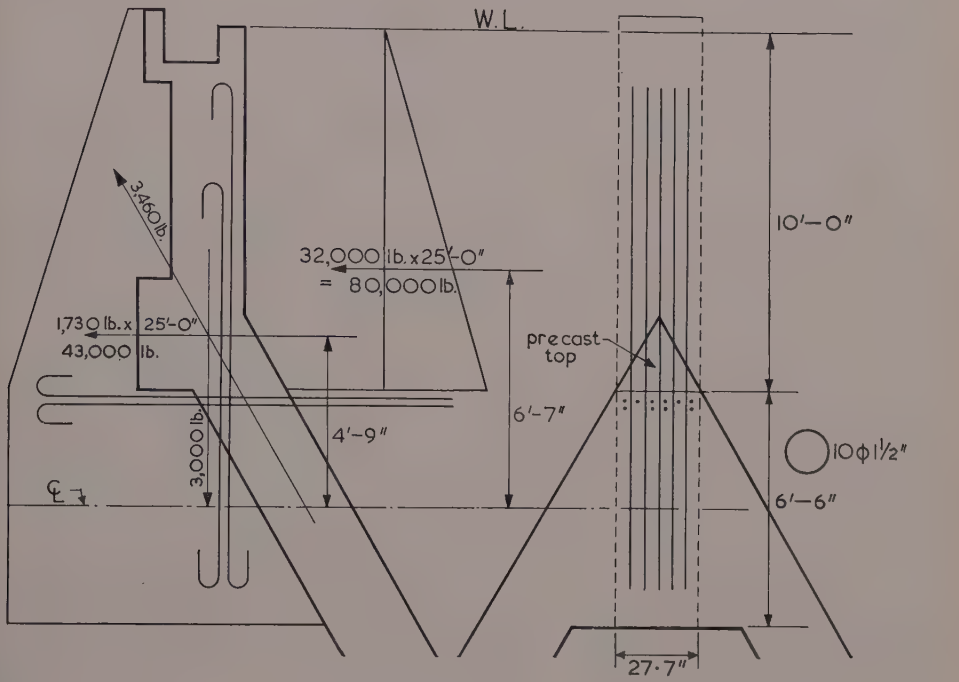


Fig. 21.7.

The vertical wall spans 26 ft. horizontally. Maximum pressure = 8 × 64 or 512 lb. per sq. ft.

$$M = 512 \times 26^2 \times \tfrac{1}{12} = 347{,}000 \text{ lb.-in.}$$

$$\text{Direct tension} = 512 \times 12{\cdot}5 = 6{,}400 \text{ lb.}$$

Using a 24-in. wall, a resistance moment of $60bd^2$ gives $60 \times 12 \times 24^2$ or 415,000 lb.-in. Stress due to direct tension = 22 lb. per sq. in.

The vertical wall weighs about 3,000 lb. per ft. This is carried to the foundation by a thrust in the sloping wall, which is inclined at 30 deg. to the vertical, as shown in *Fig.* 21.7. Resolving vertically, thrust in wall = 3,000 sec. 30 deg. or 3,460 lb. The unbalanced horizontal component of this thrust is 1,730 lb. and the lower part of the wall is thickened out to 3 ft. over a height of 3 ft. to resist this horizontal force. Fluid pressure over height of 3 ft. at an average depth of 8 ft. 6 in. = 8·5 × 64 × 3 = 1,630 lb.

$$M = (1{,}630 + 1{,}730) \times 26{\cdot}5^2 \times \tfrac{1}{12} = 2{,}360{,}000 \text{ lb.-in.}$$

A resistance-moment value of $60bd^2$ gives $60 \times 36 \times 36^2 = 2{,}800{,}000$ lb.-in. The vertical leg of the counterfort carries the pressure acting on a width of 25 ft. and

a height of 10 ft. The total pressure is $\frac{1}{2} \times 64 \times 10^2 \times 25 = 80{,}000$ lb. The counterfort also has to carry a 25-ft. run of the horizontal component of 1,730 lb. per foot, or a total load of 43,300 lb.

M on centre-line of beam $= 80{,}000 \times 6$ ft. 7 in. $= 6{,}320{,}000$ lb.-in.
$+ 43{,}300 \times 4$ ft. 9 in. $= 2{,}460{,}000$ „ „
$8{,}780{,}000$ lb.-in.

$$\frac{M}{bd^2} = \frac{8{,}780{,}000}{27{\cdot}7 \times 78^2} = 52.$$

$$\text{Direct tension} = \frac{80{,}000 + 43{,}300}{27{\cdot}7 \times 78} = 57 \text{ lb. per sq. in.}$$

Allowing for the steel, the tensile stress on the concrete will be about 300 lb. per sq. in.

$$A_{st} \text{ in top of beam} = \frac{8{,}780{,}000}{12{,}000 \times 0{\cdot}86 \times 72} = 11{\cdot}80 \text{ sq. in.}$$

$$+ \frac{123{,}300}{2 \times 12{,}000} = 5{\cdot}15 \text{ „ „}$$

$16{\cdot}95$ sq. in.

Say ten $1\frac{1}{2}$-in. diam. bars (17·67 sq. in.).

This completes the preliminary design. The preparation of the final design is now only a matter of detail. Firstly, plans and sections must be drawn to a larger scale and the total volume and total weight of the concrete must be calculated exactly and compared with the flotation. The moments, shears and direct tensions must be checked at all points. The long bars, shown in section in *Fig.* 21.7, should be welded, the steel arrangement elsewhere presenting no particular problems.

If the vertical legs of the counterforts are placed outside the tank (as shown in the left-hand side of *Fig.* 21.6) they may greatly increase the difficulties of construction. The alternative internal counterfort, shown in the right-hand side of *Fig.* 21.6 is a much simpler job.

A comparison between *Fig.* 21.2 and *Fig.* 21.6 emphasizes the difficulty and cost of constructing tanks below ground-water level in soft waterlogged ground and illustrates how much simpler it would be to build a shallow tank on 30-ft. piles than the massive tank in *Fig.* 21.6. It also raises one of the most difficult problems that an engineer has to face. In sound dry rock the design in *Fig.* 21.2 is adequate. In soft waterlogged ground when the ground-water level rises close to ground level the heavy sections in *Fig.* 21.6 are necessary to prevent flotation. These two examples are extremes and on most sites the conditions are not sufficiently good to justify the design shown in *Fig.* 21.2 and not sufficiently bad to need the heavy sections shown in *Fig.* 21.6.

If the ground is soft but the maximum ground-water level is low, the author would suggest reducing the sizes of the sections in *Fig.* 21.6 by designing them for a resistance-moment value of $100bd^2$ instead of $60bd^2$ and by raising the permissible steel stress from 12,000 to 16,000 lb. per sq. in. Some designs have been based on the assumption that the ground carries one-half of the load, the structure carrying the remainder. This is the same as carrying all the load at $120bd^2$ with a steel stress of 24,000 lb. per sq. in. Only a close and expert study of site conditions can enable a sound decision to be reached.

If the tanks are small and the foundation layer deep down, they may be sup-

ported on piles, the vertical walls acting as beams from which the hoppers are suspended and the design being similar to that of a small coal hopper (see *Fig.* 21.8). A preliminary design for an isolated conical tank 25 ft. in diameter with sides sloping at 60 deg. to the horizontal and containing fluid weighing 64 lb. per cu. ft. is shown in *Fig.* 21.9. Since the whole load has to be carried to the foundation the walls are ' load-bearing ' and should not be less than 8 in. in thickness. The value of H in *Fig.* 21.4 is 8·0 + 21·65 = 29·65 ft. The maximum hoop tension in the

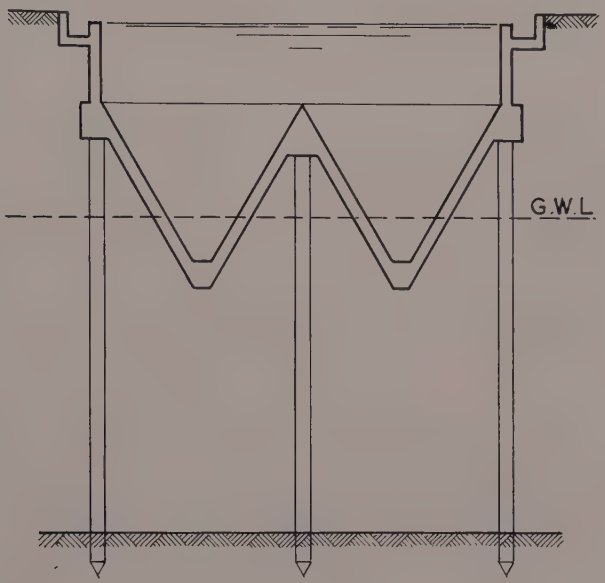


Fig. 21.8.

cone occurs at a depth of 14·83 ft., where the value of r is 9·9 ft. To the pressure of the liquid the resolved part of the self-weight must be added, which is 0·5 × 100 or 50 lb. per sq. ft.

$$p = 14{\cdot}83 \times 64 = \quad 950 \text{ lb. per sq. ft.}$$
$$+ \quad 50 \text{ ,, ,, ,, ,,}$$
$$1{,}000 \text{ lb. per sq. ft.}$$

Ring tension per foot of height on slope = 1,000 × 9·9 = 9,900 lb. On a section 8 in. by 12 in. in size, this is only 103 lb. per sq. in. The ring tension at the base of the vertical wall per foot of height is 8 × 64 × 12·5 = 6,400 lb. only. It is assumed that the ground water may rise to a point 8 ft. below the working water level in the tank.

Weight of tank above G.W.L.	= 100,000 lb.
Weight of tank below G.W.L.	= 208,000 lb.
	308,000 lb.
Weight of contents	= 478,000 lb.
	786,000 lb.

This is a load of 351 tons on a foundation 10 ft. in diameter or 4½ tons per sq. ft. The submerged volume is 4,900 cu. ft. In fresh water the uplift is 306,000 lb.,

If the assumed ground-water level already includes a factor of safety then the completed tank is safe but the incomplete lower part has a net uplift of (306,000 — 208,000) or 98,000 lb. and relief tubes must be left in the lower part of the cone in case the pumps break down before the upper part is complete. If the thickness of the upper wall is increased from 8 in. to 12 in., this would add 33,000 lb. If the thickness of the sloping wall is also increased from 8 in. to 12 in. this would add a further 46,000 lb. but would increase the flotation by 19,000 lb., giving a total weight of 387,000 lb. against a flotation of 325,000 lb., or a factor of safety of 1·2.

The stresses on Section A–A in *Fig.* 21.9 must be checked. The tank above

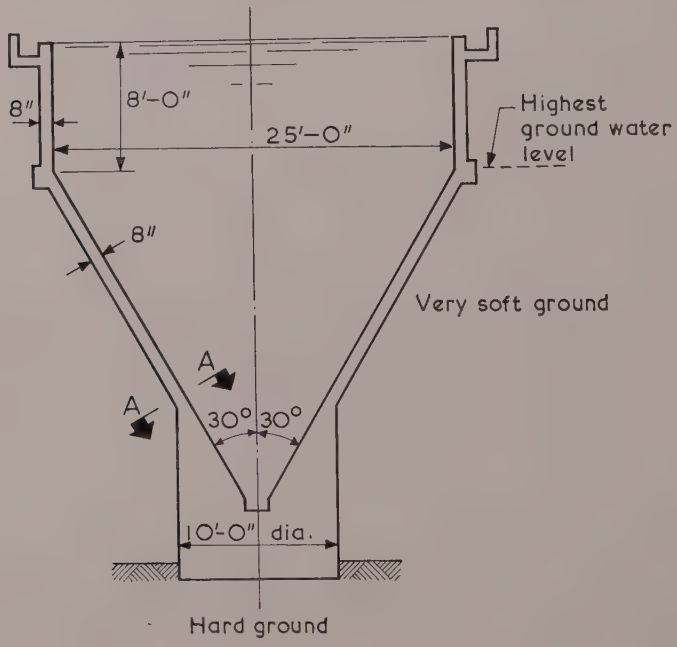


Fig. 21.9.

this section weighs 301,000 lb. and the contents standing on this part of the tank weigh 397,000 lb.

(Total thrust) cos 30 deg. = 201,000 + 397,000. Total thrust = 690,000 lb.

The section is a ring 8 in. in thickness and about 10 ft. in diameter, or about 3,000 sq. in. in area, giving a stress of 230 lb. per sq. in. Of the total flotation of 306,000 lb., some 165,000 lb. comes on the underside of the foundation and 141,000 lb. on the sloping wall, so there is still a downward load of (201,000 — 141,000) lb. on Section A–A when the tank is empty and the ground water is at its highest level.

These tanks carry a small trough round the outside. This is made the same depth all the way round and any falls that may be required are made later in mass concrete. These troughs are easily carried as shown in *Fig.* 21.10. The moment due to the eccentricity of loading is balanced by a tension ring of extra steel and a ring of additional compression in the wall at some arbitrary distance, say 2 ft. to 3 ft. below. The thickening shown at the junction of the sloping and vertical walls in *Fig.* 21.11 is to give more grip length to the bars on the inside face and to form a tension ring to resist the horizontal component of the inclined thrust. Owing

to the change in direction there is an unbalanced outward load of 762 lb. per foot, producing a ring tension of 9,530 lb. and needing an extra 0·795 sq. in. of hoop steel

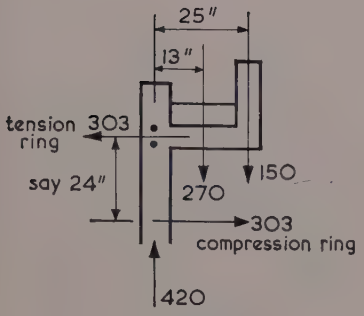


Fig. 21.10.

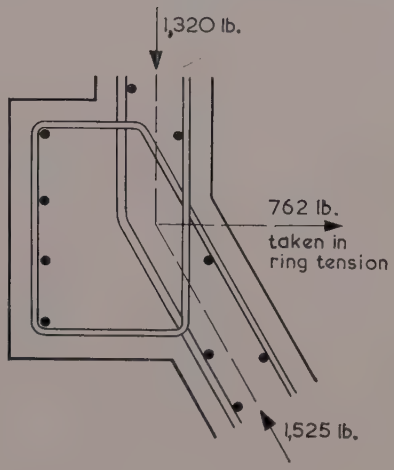


Fig. 21.11.

Construction in Soft Wet Ground

If it is possible to lower the ground-water level easily by pumping in an independent sump or large borehole, the tank may be constructed as shown in the

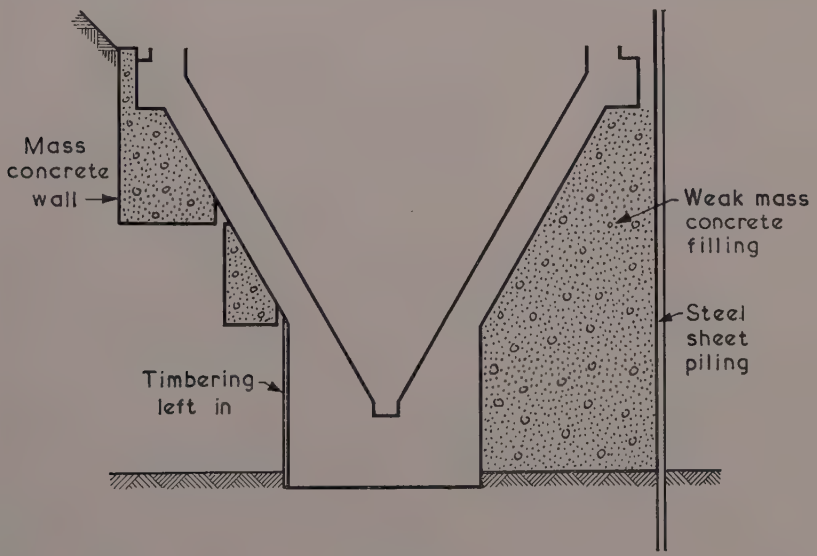


Fig. 21.12.

left-hand side of *Fig.* 21.12, provided that the dry ground is sufficiently strong to carry the mass concrete walls. The upper lifts can be built in trenches supported by timber runners (or trench sheeting), the dumpling being removed later. The

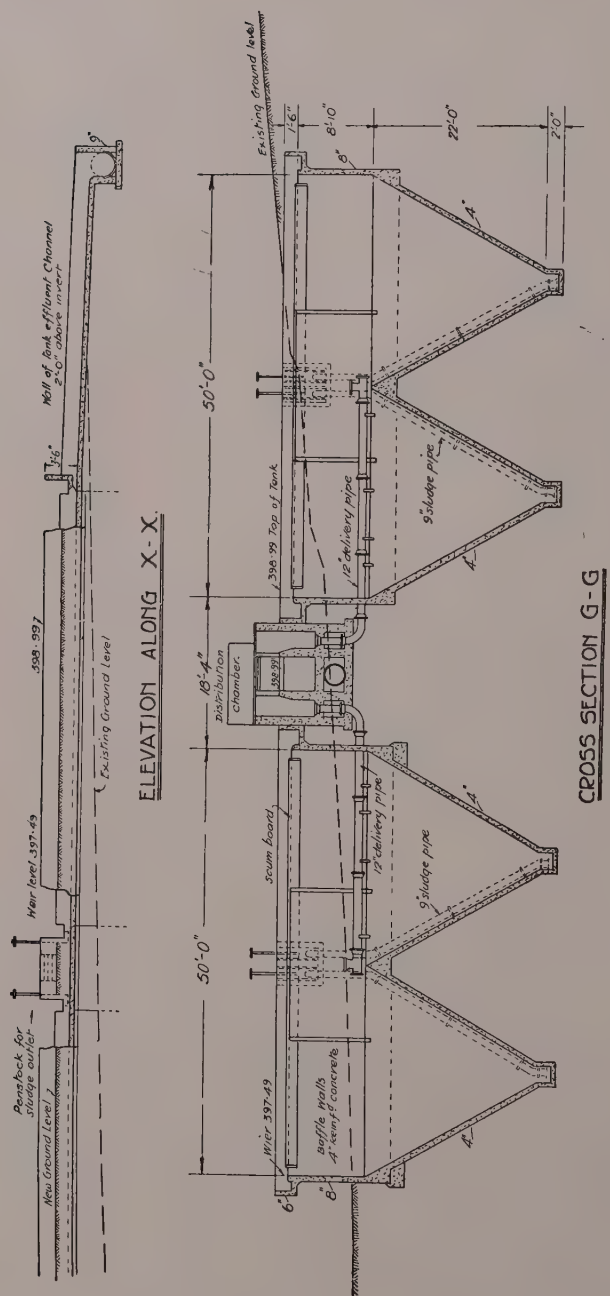

ELEVATION ALONG X-X.
Existing Ground level
Weir level 397.49
Pentock for sludge outlet
New Ground level
Wall of tank effluent Channel 2'0" above invert
Existing Ground level
CROSS SECTION G-G
50'-0"
50'-0"
18'-4"
Distribution chamber
Scum board
Weir 397.49
Baffle walls 4" thick of concrete
398.99 Top of Tank
12" delivery pipe
9" sludge pipe
12" delivery pipe
9" sludge pipe
Existing Ground level
1'6"
8'-10"
22'-0"
2'0"
6"
8"
4"
4"
4"

Fig. 21.13.

inside faces of these walls are rough-shuttered and the actual tank walls are cast against them. In the figure they are shown shuttered on the slope but they could be built in vertical lifts of about 3 ft. On the right-hand side of *Fig.* 21.12 a tank in very soft waterlogged ground needing steel sheet-piling is shown.

If many similar tanks are required the inside formwork may be simplied by supplying a structural steel framework to resist the pressures due to placing the

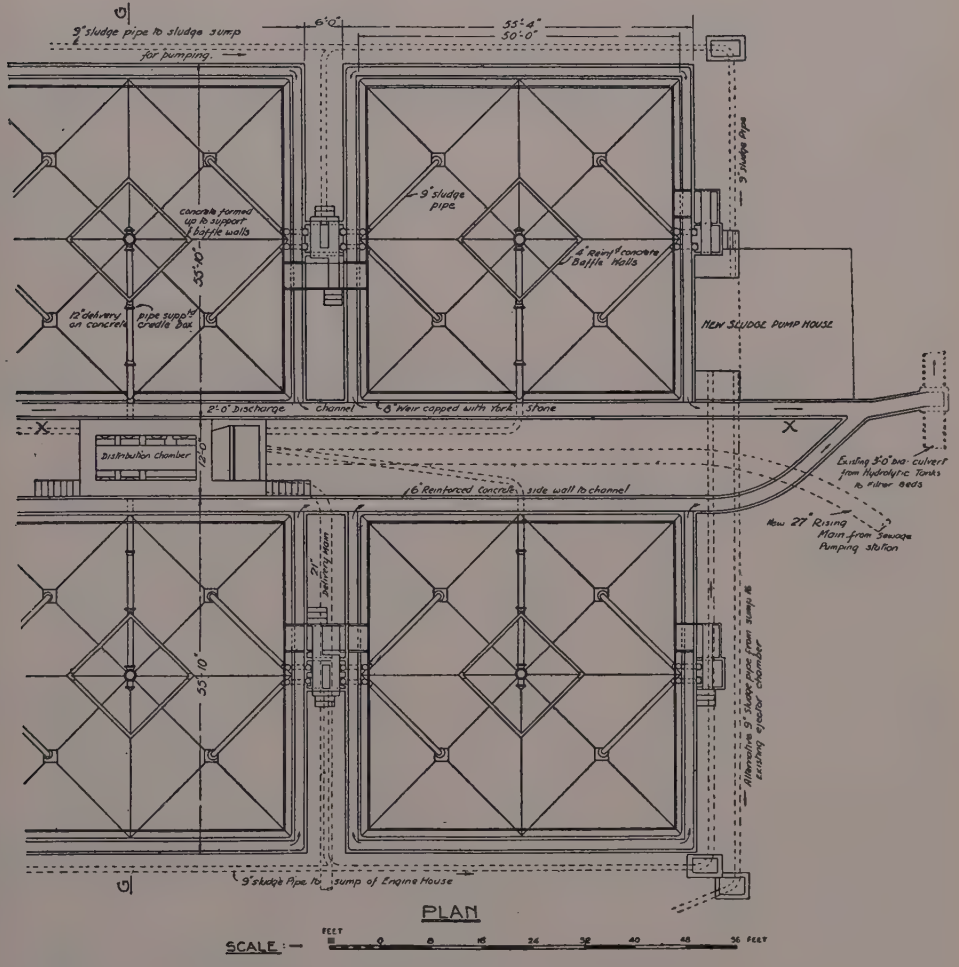

9" sludge pipe to sludge sump for pumping.
6'-0"
55'-4"
50'-0"
9" sludge pipe
9" sludge pipe
concrete formed up to support baffle walls
2'-6" Reinf.d concrete Baffle Walls
55'-10"
12" delivery pipe on concrete
pipe supp.d on cradle box
NEW SLUDGE PUMP HOUSE
2'-0" discharge channel
8" Weir capped with York Stone
X
X
Distribution chamber
12'-0"
6" Reinforced Concrete side wall to channel
Existing 3'-0" dia. culvert from Hydrolytic Tanks to Filter Beds
New 27" Rising Main from Sewage Pumping Station
27" Delivery Main
55'-10"
Alternative 9" Sludge Pipe from sump to existing ejector chamber
9" sludge Pipe to sump of Engine House
PLAN
SCALE :—
FEET
0 8 16 24 32 40 48 56 FEET

Fig. 21.14.

concrete and to ensure the correctness of the main setting-out lines. The formwork for a single isolated tank may be brought up hand-over-hand using screw sockets (see the ' Design and Construction of Formwork ' by Wynn and Manning).

Details of a set of six settlement tanks constructed at Luton are reproduced in *Figs.* 21.13 and 21.14. The upper portion of each tank is reinforced, but the pyramidal bottoms are of plain concrete bearing on chalk. Precast concrete blocks were used to form the top portions of the internal pyramid walls at their intersections.

Imhoff Tanks

In comparison with the volume of concrete to be placed during its construction, the Imhoff tank is a very costly structure due to the difficulty of placing the concrete and removing the formwork of the baffle walls. Owing to the angle of the walls with the horizontal being great, double formwork is indispensable and this must be supported from the bottom of the tank. A fairly wet concrete must be used to penetrate between the reinforcing bars, and this class of concrete requires an upper form tied down to the bars.

Recent developments in the construction of such tanks include the use of gunite instead of insitu concrete and the substitution of precast slabs for the cast-insitu work. The former permits the elimination of the top formwork and produces a slab with a high density, but the lower formwork cannot be dispensed with unless precast slabs are used. Provided that the slabs are of a size which can be economically handled and set in position there would appear to be an ultimate saving in construction costs by precasting them. Time would be saved by casting the slabs while the formwork and concreting of the walls are being executed.

Some years ago a system was tried in the United States in which the concrete

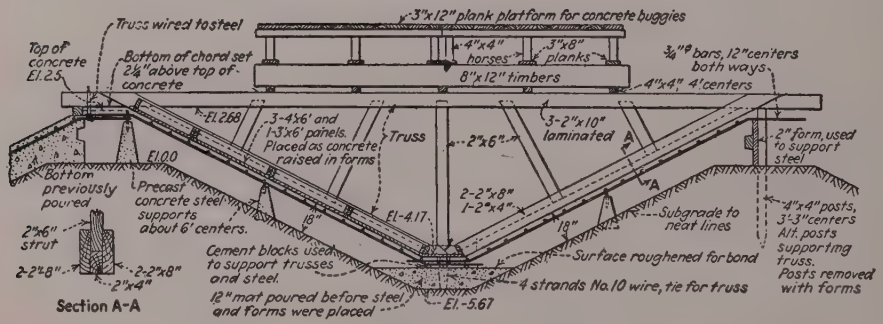

3"x12" plank platform for concrete buggies
Truss wired to steel
Top of concrete El. 25
Bottom of chord set 2¼" above top of concrete
4"x4" horses
3"x8" planks
8"x12" timbers
¾" ø bars, 12" centers both ways
4"x4", 4' centers
El. 26'8"
3-4'x6' and 1-3'x6' panels. Placed as concrete raised in forms
Truss
2"x6"
3-2"x10" laminated
2" form, used to support steel
El. 00
Bottom previously poured
Precast concrete steel supports about 6' centers.
2"x6" strut
2-2"x8"
2"x4"
Section A-A
18"
El. 4.17
2-2"x8"
1-2"x4"
Cement blocks used to support trusses and steel.
12" mat poured before steel and forms were placed
El. -5.67
Subgrade to neat lines
Surface roughened for bond
4 strands No. 10 wire, tie for truss
4"x4" posts, 3'-3" centers Alt. posts supporting truss. Posts removed with forms

Fig. 21.15.

baffles were replaced by slate panels, 1 in. in thickness, supported by reinforced concrete beams, thus avoiding formwork for the sloping panels. Another improvement in the construction of these tanks which has been adopted is to precast the V-shaped deflector beams in inverted moulds. In this way the sharp angle at the apex remains uninjured if the beam is carefully handled. In some cases the beams have been concreted in the usual manner up to a level of within 6 in. of the top and this portion has been completed by a precast coping.

A special method of constructing Imhoff tank forms is shown in *Fig.* 21.15. The side slopes are approximately 2 to 1 and required top shuttering. A 12-in. layer of concrete was first placed in the bottom and precast concrete blocks placed to support the reinforcement. After the latter had been fixed and wired together the timber trusses were set at intervals of 6 ft. 6 in. and panel forms placed between the trusses as concreting proceeded. By placing the panels on each side one at a time the ramming of the concrete was simplified. Care was taken to keep the concrete levels on both sides of the trough approximately the same. The concrete was delivered in carts on runways above the trusses and directed into place by small portable chutes.

CHAPTER TWENTY-TWO

SWIMMING POOLS

Small Private Pools

THERE are firms that specialize in the design, construction and equipment of small private swimming pools. The design may consist of a revival of the old technique of packing the ground (possibly mixed with a little cement) tight behind the wall and allowing the water pressure to pass through the wall on to the earth banking. In this particular market the additional expense of providing a wall that will carry all the water pressure with no assistance from the ground is scarcely justified. It is more a case of designing an earth bank that will remain stable in winter and summer than of designing a reinforced concrete wall. The actual wall may be of hollow concrete blocks with light steel rods passing through the holes, which are then filled solid, the inside face of the pool so formed being rendered.

Needless to say, walls of very light construction cannot be guaranteed crack-proof, but with reasonable workmanship pools of this type should give years of service.

Larger Swimming Pools

This is no place to discuss the proper length, width or depth of a swimming, water-polo or diving pool or to join in the main argument between those who regard a pool as exclusively a venue for top-class competitive sport and those who regard it as a means for improving the health and safety of the nation by teaching them the elements of swimming and watermanship.

Pools are seldom less than 2 ft. 6 in. deep. With lesser depths an adult's hands touch the bottom on front crawl. Diving pools are usually 16 ft. (5 metres) deep. Within this range of depths the walls will normally be of cantilevered slab construction (see Chapter 9).

Some deep pools have underwater lighting. Access to these lights and to the scum-channel and bath-surround drains may be provided by small isolated manholes, but a continuous gallery such as is shown in *Fig.* 22.1 is better. In this case the roof of this gallery could be used as a horizontal collar beam to support the top of the wall, but there is always the danger that someone, at some future date, may cut an opening in it for an additional access manhole and thus destroy its strength.

For a shallow tank the walls may be built before the bottom. This will make it possible for the floor to be covered with wet sand and sprayed with water without interfering with the construction of the walls. It may also allow the construction of the floor to be postponed until the weather is more favourable. It is particularly advisable in the case of an indoor bath where the floor should, if possible, be left until the building is complete. A detail is shown in *Fig.* 22.2. The reader is referred to Chapter 16 for a general discussion on tank floors.

A swimming pool may be (but seldom is) built on the upper floor of a building. The reader should then refer to Chapter 18.

The walkway around an outdoor pool may be monolithic with the walls and may be designed as a horizontal cantilever with steel near its upper surface if there is risk of its settling. A width of 3 ft. to 6 ft. is usual.

In designing the walls, the conditions are (*a*) full water pressure from the

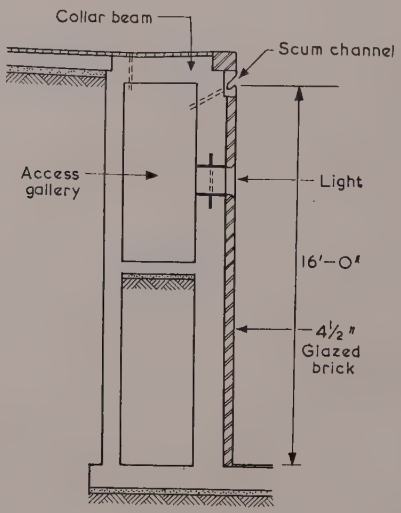


Fig. 22.1.

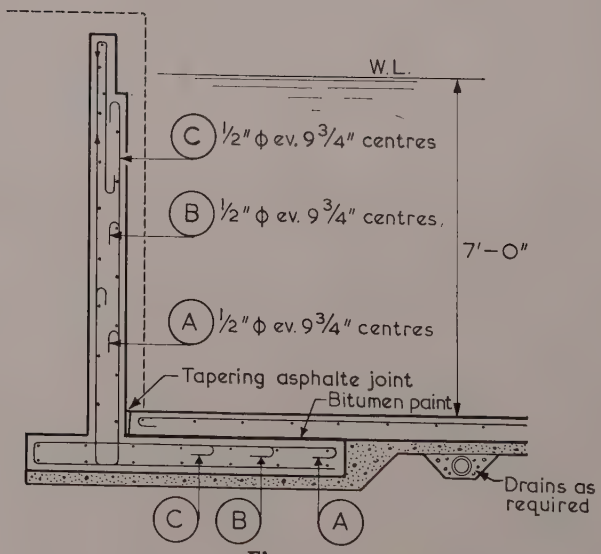


Fig. 22.2.

inside and no back pressure from the earth, except when plain concrete is used and the concrete is placed directly against the excavation; and (*b*) active earth pressure from outside when the tank is empty.

If the ground is wet and undrained, the procedure for tanks in waterlogged ground must be followed (see Chapter 20). In such cases the wall and floor may

be designed as a continuous channel section, or the floor may span longitudinally between transverse beams supported by the side walls. These beams are constructed below the floor level and must be provided with sufficient vertical links to prevent the floor slab being pushed upward and away from the beams. The links should be of closed shape, encircling the top and bottom bars in the beams, and the bottom bars in the slab should pass underneath the top bars in the beams. Provision must also be made to prevent flotation.

The scum channel may be in terrazzo-finished precast coloured concrete, but white glazed stoneware is preferable. Attempts to form a reasonably good scum channel insitu are unlikely to succeed.

A sketch of a suitable detail of the intersection of the pipework with the wall is shown in *Fig.* 22.3. The extra flange may be cast on, welded on or may be a bolted-on puddle flange.

In *Fig.* 22.4 details are shown of the reinforcement of the wall and floor of a swimming pool built at a school at Addington. The proportions used in mixing the concrete were approximately 1 : 1½ : 3, and no waterproofing compound was employed.

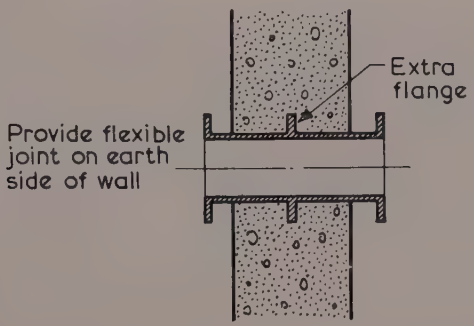


Fig. 22.3.

A cross-section of a swimming pool built at Bishop's Stortford is illustrated in *Fig.* 22.5. In this case, external water being present, an upward uniformly-distributed load averaging 200 lb. per sq. ft., was considered to be applied over the whole floor area of the empty pool. The transverse beams were carried up the walls in the form of vertical counterforts.

A method of obtaining watertightness without using a rich mix throughout a plain concrete wall consists in placing a movable steel shutter, about ⅛ in. thick, inside the forms and about 1½ in. or 2 in. from the inner face of the wall. A rich concrete is placed between this shutter and the inner form and the shutter is raised as concreting progresses to allow the two mixtures to bond together. The method is not applicable in ordinary reinforced concrete walls. For good results both concretes must be mixed to approximately the same consistency.

Expansion joints are easily made in the walls and are advisable at intervals of 30 ft. to 50 ft. Their cost is so small that it is not worthwhile taking the risk of cracks by omitting them. For more detailed information on expansion and construction joints, reference should be made to pages 100 and 174.

Structural cracks, as distinct from those caused by expansion and contraction, are more likely to occur in plain concrete walls than in reinforced concrete. These are sometimes due to uneven settlement or movement of the ground.

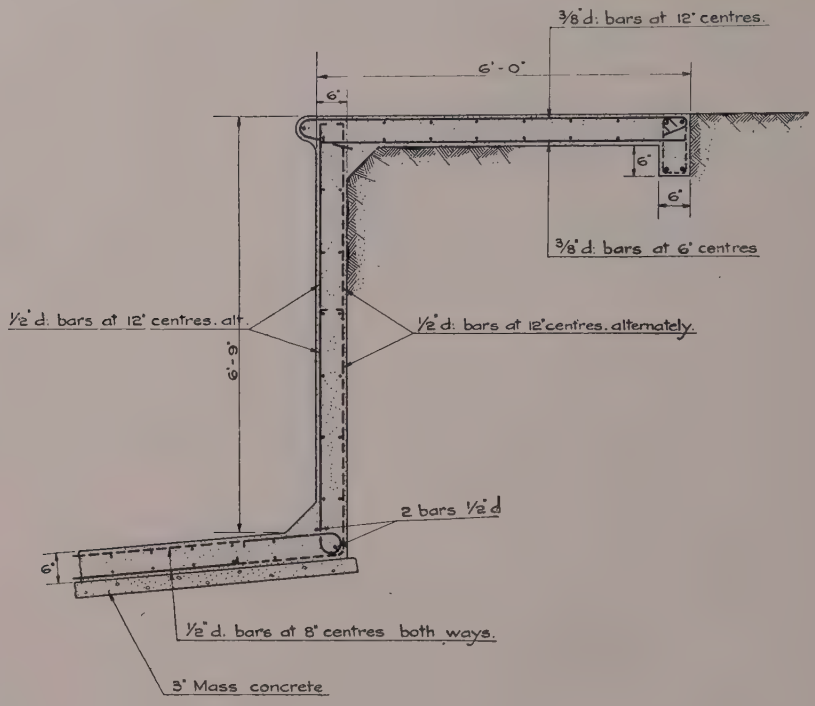


Fig. 22.4.

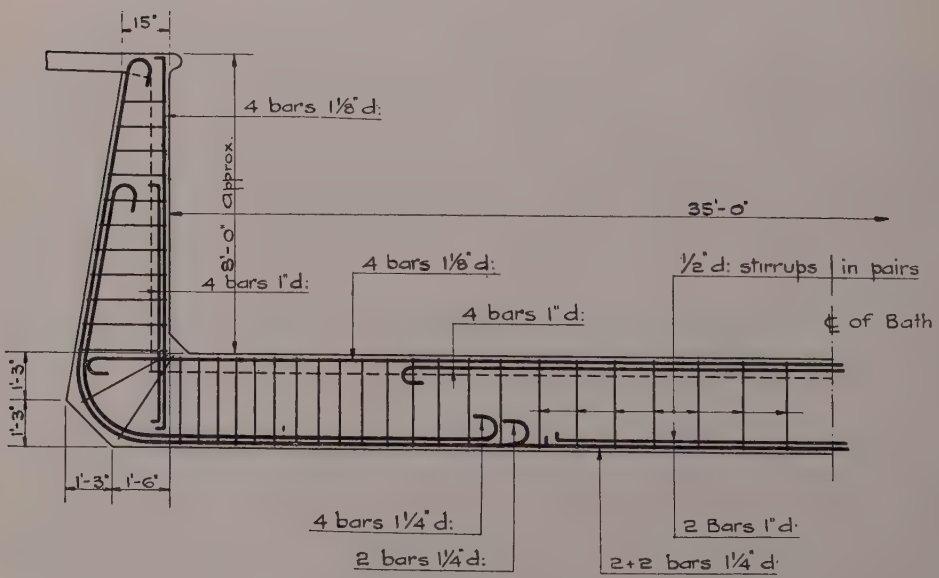


Fig. 22.5.

A suitable type of expansion joint for a plain concrete wall of a swimming pool is shown in *Fig.* 22.6. Assuming that the bay A is to be concreted first, a stop is built up between the forms and the expansion plate of 16-gauge copper, allowed to project equally on both sides of the stop. After bay A has hardened, the stop timbers are removed and the end of the wall painted with hot tar. Elastic filler, such as felt impregnated with bitumen, is then fixed against the tarred surface and painted with hot tar which is also applied to the projecting metal plate. The second bay B is then ready for concreting. This joint is an improvement on the method in which a gap is left between the bays A and B and afterwards filled with poured asphalt. When the joint is concreted in the ordinary manner it is usually found that the width is excessive. The expansion joint must pass through the coping, railings, pipes, scum channels, rendering, etc., and any facing concrete placed with a sliding shutter.

Curing the concrete in the floor of a swimming pool is not always a simple problem, particularly if the deepest part of the pool is between the ends. The difficulty is more pronounced in modern designs because of the demand for a white or coloured cement finish to the floor which entails two-course work. 'Ponding' is impossible as a rule, and the use of wet sand is an operation involving labour in

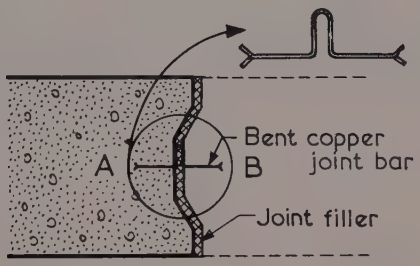


Fig. 22.6.

shifting it from place to place besides the nuisance of having sand on the floor. When the deepest part of the pool is at one end and the surfacing is to follow the base concrete it is probably best to start at the lower end and use a spray or curing compound, allowing the men to work upwards towards the dry part of the excavation. If the valley occurs between the ends of the pool, the procedure to be adopted will depend on the means available for freeing the valley from water.

In good ground the reinforcement may be placed near the top of the slab as shown in *Fig.* 22.2 so as to diminish shrinkage cracks if the bath remains empty for long.

The internal dimensions of the pool shown in *Fig.* 22.7 are 105 ft. long by 30 ft. wide. A platform 9 ft. wide surrounds the pool at the level of the top of the walls. At its outer edge there is a step 1 ft. 6 in. high leading to a second platform 3 ft. in width. Both platforms are of reinforced concrete and are 6 in. in thickness. The depth of the pool, measured from the top of the wall, is 4 ft. 6 in. at the shallow end, and the bottom is sloped at 1 in 30 for a length of 60 ft. In the next 20 ft. the depth increases to its maximum value of 9 ft. 6 in., which is retained over a length of 4 ft. 6 in. and then diminishes to 5 ft. at the end of the pool. The floor is 9 in. in thickness at the sides, its upper surfaces being sloped so as to give a fall of 3 in. from the sides to the longitudinal centre-line, and a slab thickness of 6 in. along this line. There are nine beams under the floor, dividing the floor into ten

Fig. 22.7.

Fig. 22.8.

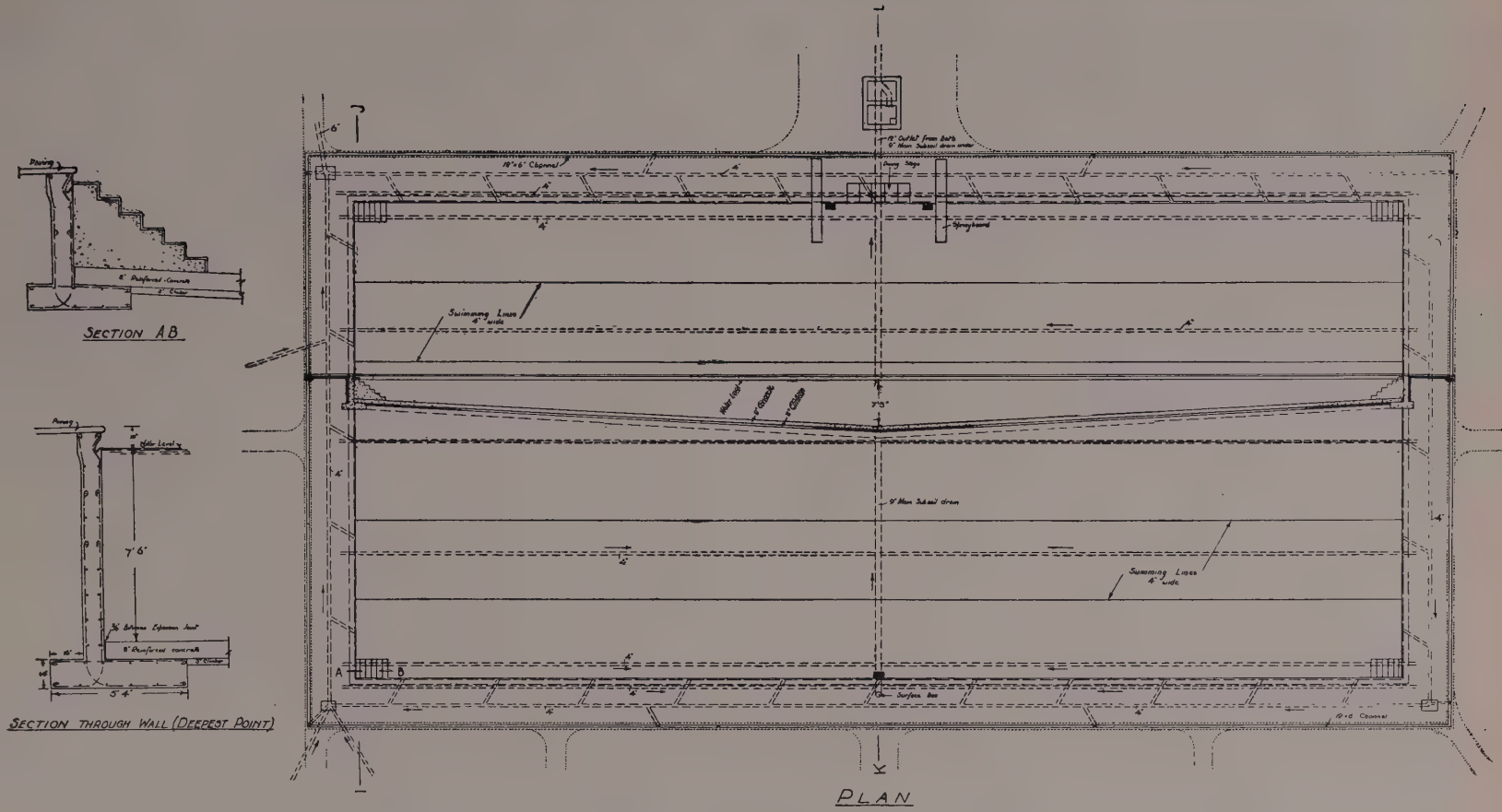
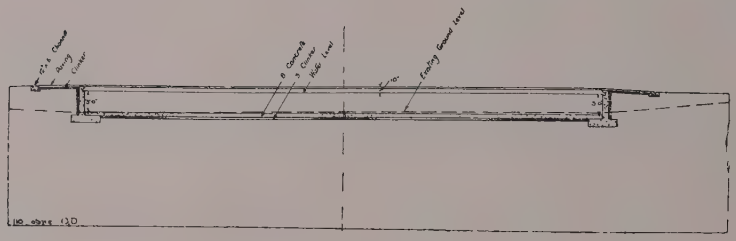
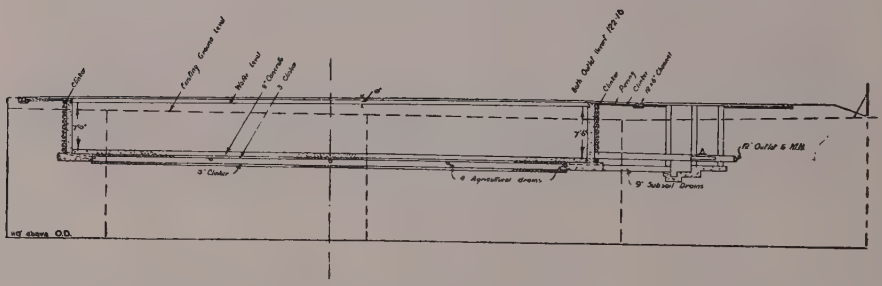

Paving
8" Reinforced Concrete
SECTION AB
Paving
Water Level
7'6"
3/4" Bitumen Expansion Joint
8" Reinforced concrete
5'4"
SECTION THROUGH WALL (DEEPEST POINT)
6"
2" Outlet from bath
9" Main Subsoil drain under
10"x6" Channel
Diving Stage
Springboard
Swimming Lines 4" wide
Water Line
1" Gutter
1" Gutter
7'6"
9" Main Subsoil drain
Swimming Lines 4" wide
A
B
Surface Box
10"x6" Channel
PLAN

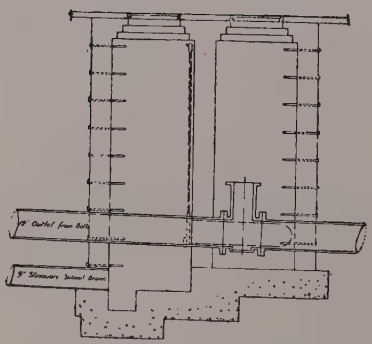
Fig. 22.9



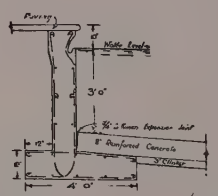
SECTION I J



SECTION K L



SECTION · THROUGH · MANHOLE



SECTION THROUGH WALL (SHALLOW END)

Fig. 22.10.

bays. Each beam is 12 in. wide, but the depths of the beams below the slab range from 20 in. to 24 in. The main reinforcement in the floor slab is near its upper surface between the beams; the latter are also reinforced near their upper surfaces at midspan and are designed to span between the side walls. The reinforced concrete side walls are 9 in. thick and provided with 9-in. by 9-in. fillets where they intersect the floor. The principal reinforcement is in a vertical direction. Horizontal bars, $\frac{3}{8}$ in. in diameter and at 12-in. centres, were fixed inside the vertical steel near both faces of the wall. The inlet and outlet pipes are 8 in. and 15 in. diameter respectively, and there is a manhole, 5 ft. square by 6 ft. 6 in. deep, monolithic with the side wall at the deepest part of the pool. The concrete proportions were 1 part of rapid-hardening Portland cement to $1\frac{1}{2}$ parts of sand and 3 parts of crushed stone, and the inside and floor were rendered with white Portland cement.

An open-air pool (*Figs.* 22.9 and 22.10) constructed for the Hornsey Borough Council is 165 ft. long by 75 ft. wide, and the depth of water ranges from 3 ft. at each end to an average of 7 ft. 3 in. along the line parallel to the shorter side of the pool and passing through its centre (see Section K–L, *Fig.* 22.9). A total of 150 single dressing-boxes is provided and five family boxes. The walls of the pool are of concrete 10 in. in thickness and reinforced as shown to resist earth pressure when the pool is empty and water pressure when it is filled. The footing slab varies from 5 ft. 4 in. to 4 ft. in width, and from 14 in. to 12 in. in thickness. Clinker has been back-filled against the walls to facilitate drainage. The subsoil drainage of the site of the pool is by means of seven rows of 4-in. tile drains leading to a 9-in. perforated subsoil main. The floor of the pool (*Fig.* 22.8) consists of 8 in. of concrete laid on 3 in. of clinker and reinforced with mats of double-mesh reinforcement.

Alternative Construction

For a shallow pool in waterlogged ground the walls may be of light steel sheet-piling faced with $4\frac{1}{2}$ in. of glazed brick in lime mortar. The floor may be in mass concrete, sufficiently thick to resist flotation, and faced with tiles or given several coats of white cement wash.

Teaching Pools

While ' Olympic size ' swimming and diving pools figure prominently on television and in the press, there is a much more real need for indoor teaching pools built exclusively for this purpose.

An excellent example is the pool constructed recently at Woollahra, Sydney, Australia. This is about 75 ft. by 40 ft. in plan and has a uniform depth of 3 ft. The floor surrounding the pool is 3 ft below water level also, so that the instructors are on a level with their pupils.

CHAPTER TWENTY-THREE

RESERVOIR ROOFS

Slab-and-Beam Roofs

Some early reservoir roofs were exact replicas of the slab-and-beam floors and roofs that were commonly used in buildings. They were often designed using permissible stresses of 600 and 16,000 lb. per sq. in. for the concrete and steel, respectively. The tensile stresses in the concrete were not calculated and were ignored. The slabs and beams were designed for moments of $\pm\frac{1}{12}wL^2$ in the internal spans and $\pm\frac{1}{10}wL^2$ in the external spans. If properly detailed and built these roofs were sound structurally but some had small nominal concrete covers (such as $\frac{1}{2}$ in. for slab bars and 1 in. for main bars in beams) and few designs or specifications called for positive means for keeping the bars in place. Often there were patches where the only protection was two coats of cement wash (which was the usual surface finish and was applied as soon as the shuttering was removed). Some roofs had deep narrow beams overcrowded with steel and no proper means was provided to prevent this steel leaning against the sides or resting on the bottom of the beam boxes. To save overall height the beams were often allowed to dip below the water level. The fact that trouble developed far too frequently due to rusting of the steel came as no surprise to anyone who was closely connected with the contracting side of the industry.

Roofs of this type are now rare. The reader may find detailed information regarding them elsewhere (see the author's 'Reinforced Concrete Design').

The one lesson to be learnt from these roofs is that reinforced concrete without proper bar spacers cannot be regarded as a permanent form of construction for exposed structures.

Mesh-panel Roofs

The author has in his possession more than a dozen sets of official bending-moment coefficients to be used in the design of mesh-panel slabs. The most severe of these documents gives coefficients which are twice as large as the least severe.

Detailed mathematical analysis of the stresses in a non-elastic, non-homogeneous, non-symmetrical reinforced concrete mesh panel is so complicated that it has never been attempted. Readers interested in the analysis of elastic plates are referred to the 'Theory of Plates and Shells' by Timoshenko. All readers are advised to study the work of Prof. H. M. Westergaard* and to pay particular attention to his dictum '. . . it would be out of the question to rely on the results of theoretical analysis only'.

Happily the reinforced concrete industry had established a standard practice for dealing with this problem before the first of the Codes of Practice appeared, and the behaviour of mesh panels designed to this standard in service and under test load leaves no doubt as to its soundness and sufficiency. Details may be found in the author's 'Reinforced Concrete Design'. Panels where the ratio of length

* Journal of the American Concrete Institute. Vol. 17, 1921. Vol. 22, 1926. Vol. 30, 1934. and Vol. 35, 1939.

to breadth exceeds 1·0 : 1·1 are best avoided, as the question becomes much more difficult both in regard to the panel itself and to the difference in stiffness between the two sets of supporting beams. There are many sets of values giving ' moment in short span ' and ' moment in long span '. Such a table appears in CP.114:1957 in which coefficients are given for ' positive moment at midspan '. All tables of this type are open to the criticism that they are a retrogression towards the early misconceptions of Grashof and Rankine, that they do not indicate that the maximum positive moments may occur across the diagonals and that no provision is made for the reinforcement to be positioned to resist these moments. This criticism is

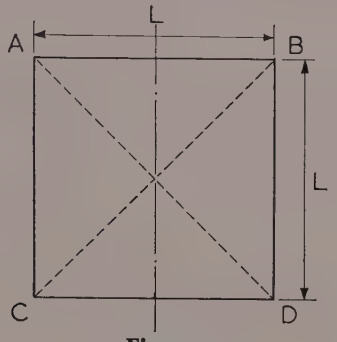


Fig. 23.1.

more serious in watertight work and is most serious when a panel has positive moments imposed on its edges instead of the normal reversed moments.

There is one set of conditions that arises in tank work that does not normally arise in the design of floors and roofs in buildings. In *Fig.* 23.1 a square isolated panel ABCD carrying a uniform load w per sq. ft. is shown. The panel is supported by four beams AB, BD, DC and CA of similar stiffness. The average unit moment on diagonals AD and BC is $\frac{1}{24}wL^2$ or $\frac{1}{24}W$, where $W = wL^2$. If, instead of an isolated panel, an internal panel surrounded by similar panels is considered, then (average reversed moment on AB, BD, DC and CA) + (average positive

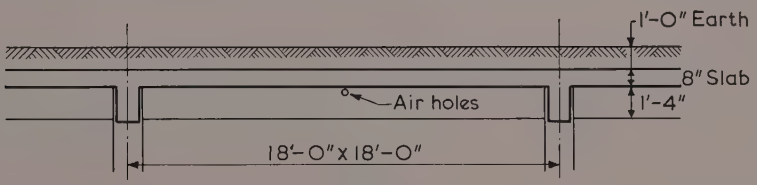


Fig. 23.2.

moment on diagonals AD and BC) $= \frac{1}{24}W$. Returning to the isolated square panel shown in *Fig.* 23.1 and applying a uniform *positive* moment M_E along the periphery of the panel, then the average positive moment on the diagonals is $\frac{1}{24}W + M_E$.

The importance of providing stiff supporting beams may be realized by referring to *Fig.* 23.1. If the stiffness of the four beams AB, BD, DC and CA is gradually reduced until finally they have no stiffness, the panel is then supported on the four points A, B, C and D and the average moment at midspan in both directions is $\frac{1}{8}W$. The condition reached is, in fact, that of an elementary flat-slab where the sum of the positive and negative moments in strips A and B is $\frac{1}{8}WL$.

A preliminary design for a mesh-panel roof with the columns spaced at 18-ft. centres in both directions is shown in *Fig.* 23.2.

Loading on slab (per sq. ft.):

Earth:	120 lb.
Imposed load:	10
Asphalt:	12
Self-weight:	96
	238 lb. per sq. ft.

$W = 238 \times 18^2 = 77{,}200$ lb.

Positive $M = \frac{1}{36}W = \dfrac{77{,}200 \times 12}{36} = 25{,}800$ lb.-in.

Negative $M = \frac{1}{36}W + 20$ per cent. $= 30{,}900$ lb.-in.
Formerly this would have been carried with a resistance-moment value of $95bd_1^2$, giving $d_1 = \sqrt{\dfrac{30{,}900}{95 \times 12}} = 5{\cdot}2$ in.

With a resistance-moment value of $60bd^2$, $d = \sqrt{\dfrac{30{,}900}{60 \times 12}} = 6{\cdot}55$ in.

If a thickness of concrete cover of $1\frac{1}{2}$ in. is necessary, an 8-in. slab with an average d_1 of 6 in. is indicated.

$$A_{st} \text{ at midspan} = \frac{25{,}800}{12{,}000 \times 0{\cdot}865 \times 6} = 0{\cdot}415 \text{ sq. in.;}$$

say $\frac{1}{2}$-in. diam. bars at $5\frac{1}{2}$-in. centres, giving 0·43 sq. in. (see *Fig.* 23.3).

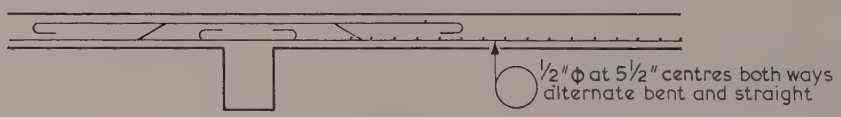


Fig. 23.3.

If one-half of the slab bars are bent up into the top of the slab over the supports, the area available to resist the reversed moment of 30,900 lb.-in. will also be 0·43 sq. in., giving a calculated stress of 13,800 lb. per sq. in. This is not exactly in accordance with the requirements of CP.2007:1960, since the slab is less than 9 in. in thickness, but this stress is quite safe, particularly if the roof is covered with asphalt. The slab has been designed to have $1\frac{1}{2}$ in. of concrete cover. This is excessive for $\frac{1}{2}$-in. diam. bars in an 8-in. slab and the design would be improved if the cover were reduced to $1\frac{1}{4}$ in.

If a support section as shown in the detail in *Fig.* 23.4 is used a beam about 12 in. wide is required.

Total load on beam $= \frac{1}{2} \times 77{,}200 =$	38,600 lb.
$+ 12$ in. $\times 16$ in. $\times 18$ ft. $=$	3,460 ,,
	42,060 lb.

Positive moment $= 42{,}060 \times 18 \times \frac{12}{12} = 757{,}000$ lb.-in.
If A_{st} provided is four $1\frac{1}{8}$-in. diam bars $= 4{\cdot}0$ sq. in.,

$$\text{lever-arm required} = \frac{757{,}000}{12{,}000 \times 4} = 15{\cdot}8 \text{ in.}$$

Then d_1 must be 18·25 in.; say beam 24 in. overall depth.
Shear $= 21{,}030$ lb. Lever-arm area $= 15{\cdot}8 \times 12 = 190$ sq. in.

Shear stress $= \frac{21{,}030}{190} = 111$ lb. per sq. in. only.

Shear resistance provided by reinforcement:

Two $1\frac{1}{8}$-in. diam. bars at 30 deg. = 12,000 lb.

$\frac{3}{8}$-in. diam. stirrups at 5-in. centres

$$= \frac{2 \times 0{\cdot}11 \times 12{,}000 \times 0{\cdot}865 \times 21{\cdot}25}{5} = 9{,}700 \text{ ,,}$$

21,700 lb.

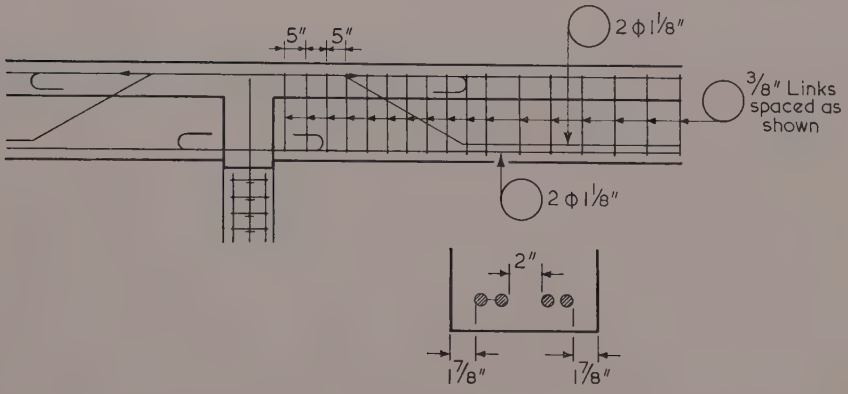


Fig. 23.4.

The column size could be 12 in. by 12 in., as shown in *Fig.* 23.4, but the joint would be sounder with a 14-in. by 14-in. column as shown in *Fig.* 23.2.

Generally speaking any roof that can be divided into a number of square panels suitable for mesh-panel construction would also be suitable for flat-slab work and modern practice would tend to adopt the latter form of construction.

Sunk-panel Flat-slab Roofs

If the requirements of CP.2007:1960 must be observed the calculated tensile stress in the concrete on the underside of the roof must be restricted to about 270 lb. per sq. in. In flat-slab work a high concentration of moment occurs over the column. This can easily be dealt with as stresses of 1,200 and 18,000 lb. per sq. in. may be employed. The critical section, if the demands of this Code must be met, is the midspan section of Strip A where the value of the resistance-moment factor must be kept to about $52bd^2$. With a standard flat-slab this necessitates the thickening of the whole slab to provide the necessary resistance at this section. With a sunk-panel design this section is already thickened.

In normal flat-slab buildings it is always possible to rely on the columns to stiffen and strengthen the floor against unsymmetrical loading. In reservoir work the columns carry comparatively little load and are usually about twice as high as a normal floor height in a building. In order to use them structurally in conjunction with the floor, it is generally necessary to increase their diameter beyond that needed to carry the direct load.

There are two main alternatives. Firstly, the columns may be made the minimum diameter necessary to carry the direct load, and the roof designed for moments

of $-\frac{1}{22}WL$, $+\frac{1}{30}WL$ and $\pm\frac{1}{60}WL$. Alternatively, the columns may be stiffened and the roof designed for moments of $-\frac{1}{28}WL$, $+\frac{1}{40}WL$ and $\pm\frac{1}{72}WL$. In this second alternative the diameter of the column must be neither less than twice the thickness of Strip A at midspan nor less than one-tenth of the clear height under the cap, and in addition the column must be sufficiently strong to carry a moment of $\frac{1}{32}W_LL$, where W_L is the total imposed load on the panel. This last stipulation raises the question as to what should be termed the imposed load on the roof. Most roofs support a foot or so of soil spread over them. Once this is in place over the whole roof it becomes a uniform dead load but while it is being placed or removed (unless it is placed and removed in thin layers) it is an imposed load within the meaning of W_L.

Assuming that the calculated tensile stress on the concrete must not exceed about 270 lb. per sq. in., approximate thicknesses for the roof may be found from the following:

(*a*) With Flexible Columns: Strip A: $\frac{L\sqrt{w}}{290}$ in.

Strip B: $\frac{L\sqrt{w}}{470}$ in.

(*b*) With Stiff Columns: Strip A: $\frac{L\sqrt{w}}{335}$ in.

Strip B: $\frac{L\sqrt{w}}{515}$ in.

where L is the panel length in inches and w is the total load per sq. ft.

To illustrate the two alternatives, consider a roof panel 18 ft. by 18 ft. carrying one foot of earth over a reservoir 20 ft. deep (see *Fig.* 23.5).

Load per sq. ft.:

Earth, say:	120 lb.
Imposed load:	10 lb.
Asphalt:	12 lb.
Estimated self-weight:	150 lb.
say	300 lb.

Total panel load $W = 18 \times 18 \times 300 = 97{,}200$ lb.

(*a*) With Flexible Columns: Strip A: $\frac{(18 \times 12)\sqrt{300}}{290} = 12{\cdot}9$; say 13 in. thick.

Strip B: $\frac{(18 \times 12)\sqrt{300}}{470} = 7{\cdot}96$; say 8 in. thick.

A 12-in. diameter column will suffice to carry the load of 97,200 lb. (43·4 tons.)

(*b*) With Stiff Columns: Strip A: $\frac{(18 \times 12)\sqrt{300}}{335} = 11{\cdot}2$; say $11\frac{1}{2}$ in. thick.

Strip B: $\frac{(18 \times 12)\sqrt{300}}{515} = 7{\cdot}26$; say $7\frac{1}{2}$ in. thick.

The clear height of the column will be about 18 ft. so the diameter is fixed at twice the depth of Strip A or $2 \times 11{\cdot}5 = 23$ in.

The preliminary designs for the two cases (*a*) and (*b*) are shown in *Fig.* 23.5. While the earth covering is being placed or removed the imposed load is (120 + 10) lb. per sq. ft.

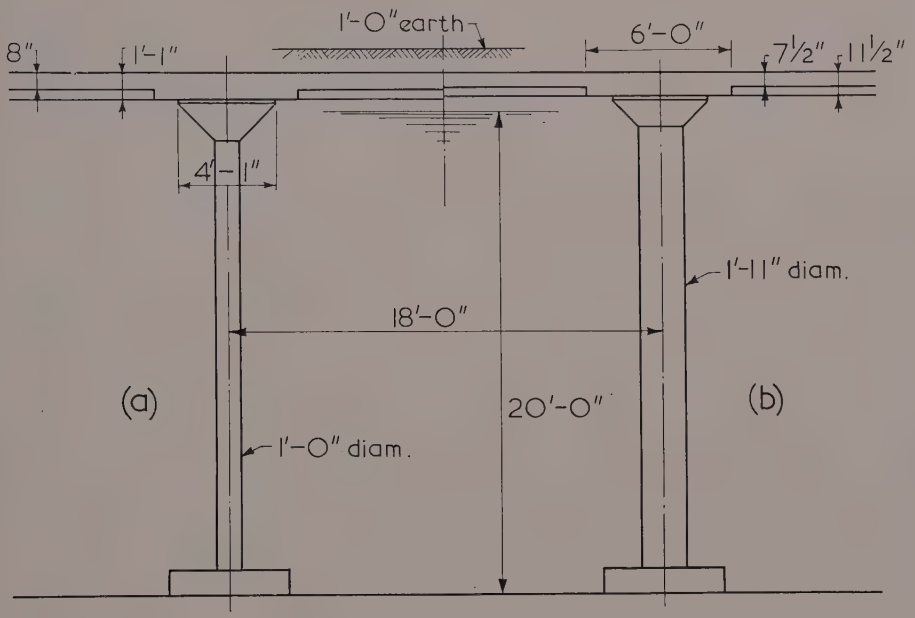


Fig. 23.5.

$$\frac{W_L L}{32} = \frac{(120 + 10) \times 18 \times 18 \times 18 \times 12}{32} = 285{,}000 \text{ lb.-in.}$$

The condition of loading is shown in *Fig.* 23.6.

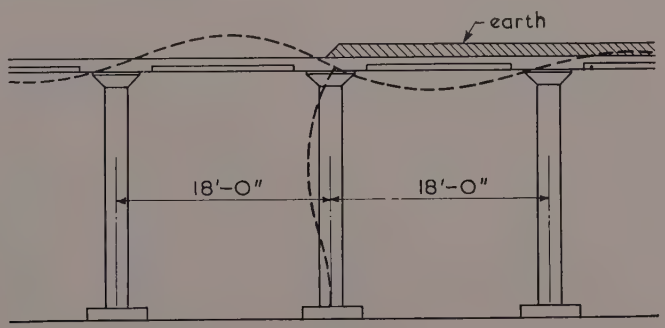


Fig. 23.6.

Load on column = 76,100 lb. Eccentricity $e = \frac{285{,}000}{76{,}100} = 3{\cdot}74$ in.

$$\frac{e}{D} = \frac{3{\cdot}74}{23} = 0{\cdot}163.$$

With one per cent. of reinforcement the maximum compression (from tables) is only 380 lb. per sq. in. The midspan sections of Strip A are shown in *Fig.* 23.7.

With flexible columns, $M = \frac{WL}{30} = \frac{97{,}200 \times 18 \times 12}{30} = 700{,}000$ lb.-in.

$$A_{st} = \frac{700{,}000}{12{,}000 \times 0{\cdot}9 \times 11{\cdot}0} = 5{\cdot}9 \text{ sq. in.}$$

With stiff columns, $M = \frac{WL}{40} = 525{,}000$ lb.-in.

$$A_{st} = \frac{525{,}000}{12{,}000 \times 0{\cdot}9 \times 9{\cdot}5} = 5{\cdot}12 \text{ sq. in.}$$

The section modulus of this latter section, taking the full concrete section and the steel into account, is 2,070 in.^{3} for tension at bottom.

Calculated tensile stress on concrete $= \frac{525{,}000}{2{,}070} = 254$ lb. per sq. in.

The self-weight of the roof has been estimated as 150 lb. per sq. ft. whereas it is only 135 lb. for the heavy roof and 122 lb. for the lighter one. Against this, it is

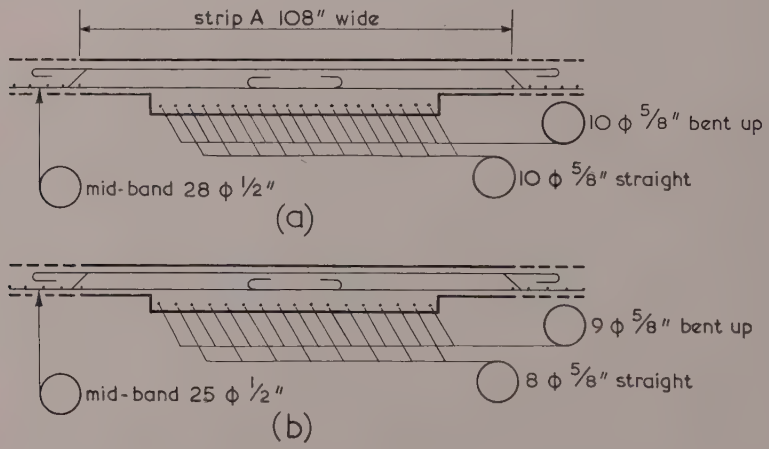


Fig. 23.7.

difficult to estimate the maximum weight the roof may have to carry when the earth is covered with long grass and saturated with rain.

It may be held that both designs in *Fig.* 23.5 are unduly pessimistic and that the thinner roof may be used in conjunction with the lighter columns. If this is done the worst that can happen is that, for the few days while the earth covering is being placed (or removed), the calculated stress in the steel at the bottom of Strip A may rise from 12,000 to 16,000 lb. per sq. in. (no practical designer would worry about this point), and that the calculated stress in the steel over the column may rise from 18,000 to 22,900 lb. per sq. in. or from 16,000 to 20,400 lb. per sq. in. (generally speaking the author is not in favour of a working stress of 18,000 lb. per sq. in. with plain mild steel bars used in watertight work).

The method of placing the earth on the roof must be clearly specified and shown on the drawings. The contractor must not be allowed to dump heaps of earth 5 ft. or 6 ft. high before spreading.

For average work in Great Britain the author would suggest using the lighter roof with 15-in. diam. columns, each reinforced with eight $\frac{3}{4}$-in. diam. bars. In some

under-developed countries both the heavy roof and heavy column should be used. The reader must carefully study the site conditions before making a final decision. In office buildings generally, the closer the spacing of the columns the cheaper the building, but a very close column spacing looks ugly and greatly detracts from the efficient functioning of the building. In a reservoir neither of these considerations, apply, although the columns are generally twice as high as an average floor height and are therefore relatively more expensive, and the provision of too many columns reduces the effective capacity of the reservoir. Spans of about 15 ft. by 15 ft. are common.

If the edge of the roof is supported on, but not structurally attached to, the top of the wall, the external panels should be 0·75 times the internal panel length. If the edge of the roof cantilevers beyond the last line of column this cantilever should be about 0·4 times the panel length.

The quantities per panel for alternatives (*a*) and (*b*) as drawn in *Fig.* 23.5, including the column are:

(*a*) 12·2 cu. yd. of concrete and 20·3 cwt. of steel bars.

(*b*) 12·1 cu. yd. of concrete and 20·0 cwt. of steel bars.

The cost of shuttering plus scaffolding for the two schemes will be almost the same. If the designer is prepared to accept the lighter roof in scheme (*b*) with 15-in. diam. columns, the quantities are 11 cu. yd. and 19 cwt. The column and column cap in *Fig.* 23.5*a* displace 153 gallons, while those shown in *Fig.* 23.5*b* displace 392 gallons. Using a 15-in. diam. column the loss of capacity is 236 gallons per column. The gross volume of a reservoir panel 18 ft. by 18 ft. by 20 ft. deep is 40,500 gallons.

Elementary Flat-slab Roofs

In *Fig.* 23.8 a preliminary design is shown for an interior panel of roof designed as an elementary flat-slab without capitals and drop panels and with no stiffness in the columns. The ruling moment is the positive moment in Strip A (tension on

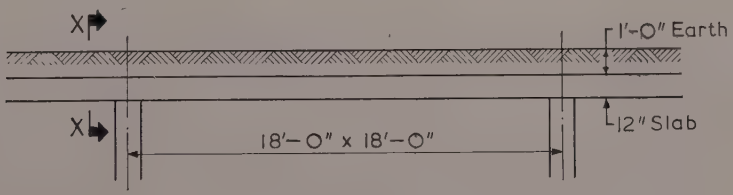


Fig. 23.8.

water face) of $\frac{1}{24}WL$, giving a unit moment of $\frac{1}{12}WL$. With a resistance-moment value of $60bd^2$, $d = \sqrt{\frac{W}{12} \times \frac{1}{60}} = \sqrt{\frac{wL^2}{720}}$. If $w = 350$ lb. per sq. ft. and $L = 15$ ft., $d = \sqrt{\frac{350 \times 15^2}{720}} = 10{\cdot}45$ in.

In *Fig.* 23.9 values are given of the imposed load per sq. ft. that can be carried by various thicknesses of slab over a given span. If the imposed load is 142 lb. per sq. ft. and if the columns are spaced at 18-ft. centres in both directions, then from *Fig.* 23.9 the slab thickness required is about 11·5 in.; say 12 in. in practice. Checking this:

$w = 142 + 150 = 292$ lb. per sq. ft. $W = 292 \times 18^2 = 94{,}600$ lb.

Width of Strip A and Strip B = 9 ft. = 108 in.

Moments in Strip A:

$$-\frac{WL}{18} = -\frac{94{,}600 \times 18}{18} \times 12 = -1{,}135{,}000 \text{ lb.-in.}$$

$$+\frac{WL}{24} = +\ 850{,}000 \text{ lb.-in.}$$

Moments in Strip B:

$$\pm\frac{WL}{60} = \pm\ 340{,}000 \text{ lb.-in.}$$

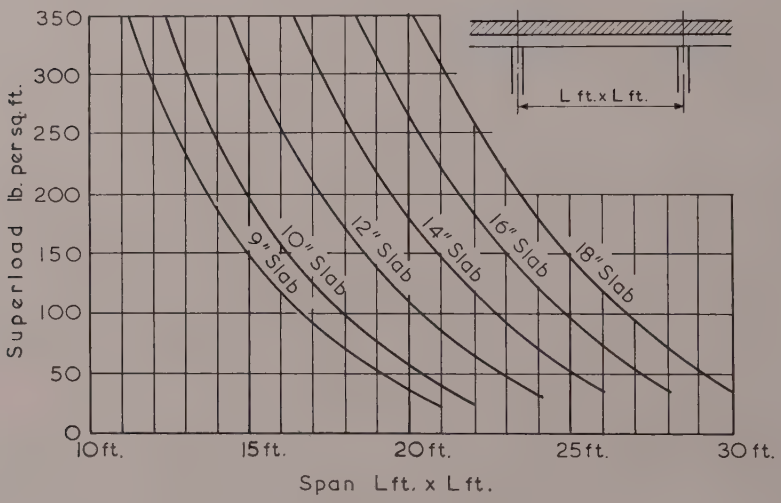


Fig. 23.9.

With resistance-moment value of $60bd^2$, $d = \sqrt{\dfrac{850{,}000}{60 \times 108}} = 11{\cdot}45$ in.

With resistance-moment value of $250bd_1^2$, $d_1 = \sqrt{\dfrac{1{,}135{,}000}{250 \times 108}} = 6{\cdot}47$ in.

A_{st} for Strip A $= \dfrac{1{,}135{,}000}{18{,}000 \times 0{\cdot}865 \times 9{\cdot}75} = 7{\cdot}48$ sq. in. (over column).

and $\dfrac{850{,}000}{12{,}000 \times 0{\cdot}865 \times 9{\cdot}75} = 8{\cdot}4$ sq. in. (at midspan).

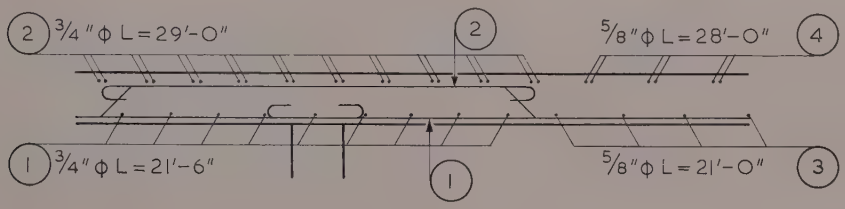
A_{st} for Strip B $= \dfrac{340{,}000}{12{,}000 \times 0{\cdot}865 \times 9{\cdot}75} = 3{\cdot}36$ sq. in.

0·3 per cent. of 108 in. by 12 in. $= 3{\cdot}88$ sq. in.

The steel arrangement is shown in *Fig.* 23.10. Each column-head band consists of 9 straight bars marked (1) and 10 bent-up bars marked (2). Each mid-panel band consists of 7 straight bars marked (3) and 6 bent-up bars marked (4). Two column-head bands plus two mid-panel bands plus four $\frac{3}{8}$-in. diameter distributors 10 ft. long complete the reinforcement in one interior panel.

The total load on the column, including its self-weight will be about 100,000 lb.

and a 12-in. by 12-in. section will carry this. The punching shear strength at the head of the column at 200 lb. per sq. in. is $200 \times 4 \times 12 \times 12 = 115{,}000$ lb. Drawing lines at 45 deg. as in *Fig.* 23.11, the lever arm is $0{\cdot}865 \times 9{\cdot}75 = 8{\cdot}45$ in. Beam shear at 100 lb. per sq. in. is $4 \times 36 \times 8{\cdot}45 \times 100 = 121{,}500$ lb.



SECTION X–X

Fig. 23.10.

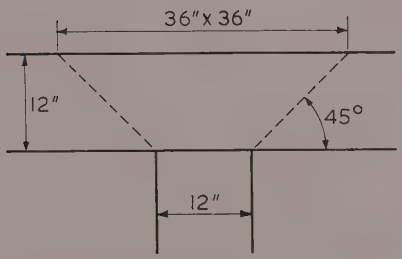


Fig. 23.11.

The shearing strength is obviously the weakest part of the design and the size of the column should be increased to 14 in. by 14 in.

Edge Details

For a small reservoir, on a site where the air temperature has a limited range (as on some tropical islands) or when the reservoir is exceptionally well insulated the edge of the roof may be connected structurally to the top of the wall.

If the designer has no objection to sliding joints, the edge of the roof may rest on the top of the wall with a sliding joint between and slide in and out as the air temperature falls or rises, the joint being bituminous material, stainless steel, synthetic fabric, multi-layer rubber, etc.

A much sounder arrangement is to cantilever the edge of the roof out over the edge of the wall. This is generally simple with a square or rectangular reservoir, but with a circular reservoir the edge distances vary continuously and several different column spacings may have to be tried before the outer bays fall into a convenient pattern.

If the wall is fairly thick, the top of it may be cantilevered inward as shown in *Fig.* 23.12, thus reducing the overall diameter of the roof. If this cannot be done, then the edge of the roof must overlap the top of the wall as shown in *Fig.* 23.13.

A preliminary design for the roof of a tank 150 ft. in diameter is shown in *Figs.* 23.13 and 23.14. The roof carries an imposed load of 140 lb. per sq. ft. and is divided into panels 15 ft. by 15 ft. in size. From *Fig.* 23.9 a 9-in. slab is required

for an elementary flat-slab design. The slab is thickened out to 12 in. to form a circular beam 36 in. wide, as shown in *Figs.* 23.13 and 23.14. This beam picks up a strip of roof about 9 ft. wide.

$$\begin{aligned} \text{Load} &= 9 \times (140 + 112) = 2{,}270 \text{ lb. per ft.} \\ &+ 3 \text{ in.} \times 36 \text{ in.} = \underline{\quad 112 \text{ ,, ,, ,,}} \\ &= 2{,}382 \text{ lb. per ft.} \end{aligned}$$

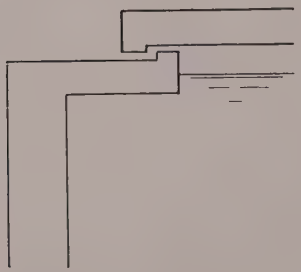


Fig. 23.12.

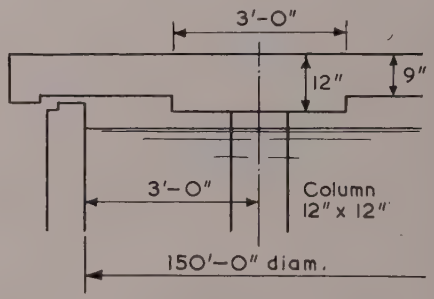


Fig. 23.13.

The maximum span is about 16 ft.

Positive moment, say $\dfrac{2{,}382 \times 16^2 \times 12}{24} = 306{,}000$ lb.-in. $\dfrac{M}{bd^2} = \dfrac{306{,}000}{36 \times 12^2} = 59.$

Reversed moment, say 612,000 lb.-in. $\dfrac{M}{bd_1^2} = \dfrac{612{,}000}{36 \times 9{\cdot}5^2} = 189.$

$$\text{Bottom steel} = \frac{306{,}000}{12{,}000 \times 0{\cdot}865 \times 9{\cdot}5} = 3{\cdot}1 \text{ sq. in.}$$

$$\text{Top steel} = \frac{612{,}000}{18{,}000 \times 0{\cdot}865 \times 9{\cdot}5} = 4{\cdot}15 \text{ sq. in.}$$

The design is workable as drawn but would be improved if the depth of the beam were increased from 12 in. to 14 in.

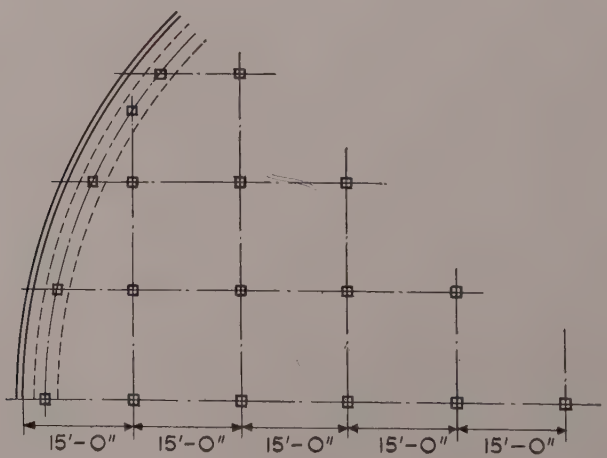


Fig. 23.14.

A circular reservoir can always be neatly roofed in with an aluminium dome (see *Fig.* 27.34).

Expansion Joints in Roofs

When the roof is independent of the wall and is supported on high slender columns, joints will not normally be required for any roof less than 200 ft. wide, particularly if the roof is heavily loaded, as this keeps down the tensile stress in the columns.

With a reservoir of width $2R$ supported on circular columns of height H and diameter d, if the heads of the outermost columns are pushed over a distance x by expansion due to temperature the primary moment in these columns caused by the movement is $6EKx/H$ or $6EIx/H^2$.

If the column carries a direct load which causes a stress of $+500$ lb. per sq. in. and a moment which gives rise to a stress of ± 800 lb. per sq. in. the stress ranges from $+1{,}300$ to -300 lb. per sq. in.

$M = 800Z = \dfrac{6EIx}{H^2}$ also. Since $Z = \dfrac{2I}{d}$, $\dfrac{400}{d} = \dfrac{6Ex}{H^2}$.

If $H/d = 15$ (a limit that is usually accepted), $E = 3 \times 10^6$ lb. per sq. in., and $x = 30 \times 6 \times 10^{-6} \times R$ (that is a rise of 30 deg. F.), then $R = 7{\cdot}42H$, and if $H = 20$ ft., then $R = 148$ ft.

The reservoir could be 296 ft. diam. or 296 ft. on the diagonal, say 200 ft. square. This result is not appreciably affected by the amount of steel in the column, since this increases both the moment and the section modulus by the same amount.

If the edge of the roof is structurally attached to the top of the wall, then any expansion joints in the wall must be continued across the roof.

Small Circular Roofs

In a small circular tank the roof may span from wall to wall without intermediate supports. It may then be designed as a mesh panel. The calculation of the moments in a uniform circular plate under symmetrical loading is a standard theorem. Unfortunately, roofs of this type are invariably provided with an access manhole which must greatly modify these standard values.

To calculate these standard values it is necessary to take into account the value of Poisson's ratio η. The value of this for reinforced concrete is sometimes assumed to be 0·25, but the average of experimental results is about 0·15. In CP.114:1957 a value of η of 0 is recommended for the analysis of reinforced concrete slabs spanning in two directions.

The principal values of the radial moment M_r and tangential moment M_t on circular slabs loaded uniformly over the whole of their area and supported at the circumference are:

Slab freely-supported at the Circumference:

The maximum values, at the centre of the slab, are $M_r = M_t = + \dfrac{3 + \eta}{16\pi} W$.

SLAB FIXED AT THE CIRCUMFERENCE:

At the centre of the slab:

$$M_r = M_t = + \frac{1 + \eta}{16\pi} . W.$$

At the circumference:

$$M_r = - \frac{W}{8\pi} \text{ and } M_t = - \frac{W}{8\pi} . \eta.$$

For a reinforced concrete slab provided with a manhole not exceeding 24 in. by 18 in. a ruling moment of $\frac{1}{18} wD^2$ may be used, where w is the load per sq. ft., D is the diameter in feet and the moment is in lb.-ft. per foot width of slab.

If the tank is 20 ft. in diameter and the roof carries an imposed load of 50 lb. per sq. ft. the value of w (assuming a 7-in. slab) is (50 + 87) = 137 lb. per sq. ft.

$$M = \frac{wD^2}{18} = \frac{137 \times 20^2 \times 12}{18} \text{ lb.-in.} = 36{,}500 \text{ lb.-in. per foot width.}$$

For a resistance-moment value of $60bd^2$, the minimum d is 7·12 in.

If the imposed load is unlikely to occur (and an imposed load of 50 lb. per sq. ft. on a circle 20 ft. in diameter gives a total of 15,700 lb. or 7 tons), a 7-in. slab is ample.

CHAPTER TWENTY-FOUR

MINING SUBSIDENCE

A VERY difficult problem has to be faced when a tank is to be built on ground which is subject to subsidence due to mining operations, a fairly common state of affairs at iron works and collieries.

It is usual to find that records have to be kept showing the settlement per annum and, if this is so, they are of great assistance in preparing the design. Where much settlement is expected in the future, a tank of large area without internal walls or a plain concrete structure is generally out of the question. The choice is then between providing a number of small tanks, a narrow tank with many cross-walls, or a series of cells which may be connected by a system of valves. No definite rules can be laid down, since each case presents difficulties of its own, but the general principles are to arrange the reinforcement in the walls in such a manner that the loads of each compartment can be carried by its walls acting either as simple beams supported at their ends or as cantilevers supported near the middle of the wall. The best procedure is to draw a plan showing the cells and to try to visualize what will happen if the ground settles below the walls at their ends or centres, or if it settles below the floor slabs. The whole problem is exceedingly difficult and a safe

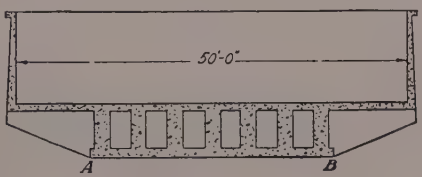


Fig. 24.1.

design is very expensive, though less costly in the long run than one which fails. The latter can seldom be patched up, since settlement is generally progressive once it begins. Careful attention is required to ensure that the loads transmitted to the ground under any of the assumed settlement conditions are not excessive at points which do not subside.

In the simplest case, where the diameter of the tanks does not exceed 50 ft., it is possible to dispense with the internal partition walls by designing the substructure with two rows of ribs at right-angles forming a cellular structure (*Fig.* 24.1). Before settlement occurs the soil resistance is equal to the weight of the structure, including that of the water, divided by the area of the base AB, and if the latter is reinforced in two directions the upward reaction at one rib is obtained by multiplying the spacing of the ribs by one-half of the unit soil pressure. This is to be combined with the local downward load to obtain the resultant load. In addition, portions of each rib act as cantilevers, each loaded with a portion of the weight of the wall and the weight of water and concrete directly above it.

Settlement may result in soil pressures and downward loading of the types shown in *Fig.* 24.2, but an unsymmetrical arrangement is also possible in which the earth sinks away from below the slab AB along a line cutting a triangular area

from the square base. The effects of unsymmetrical settlement are somewhat difficult to estimate; in practice it has been found that an assumption of settlement of the ground over an area of not less than one-third that of the base AB is sufficient in most cases.

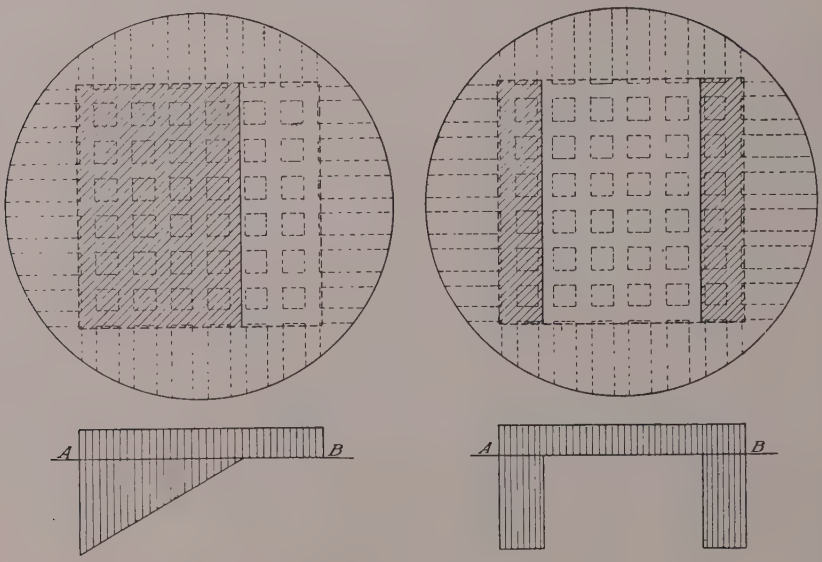


Fig. 24.2.

Extracts from a report on subsidence due to mining as it affects reservoirs is given at the end of this chapter (page 275).

Examples.—A service reservoir was constructed at Essen in two parts (*Fig.* 24.3)

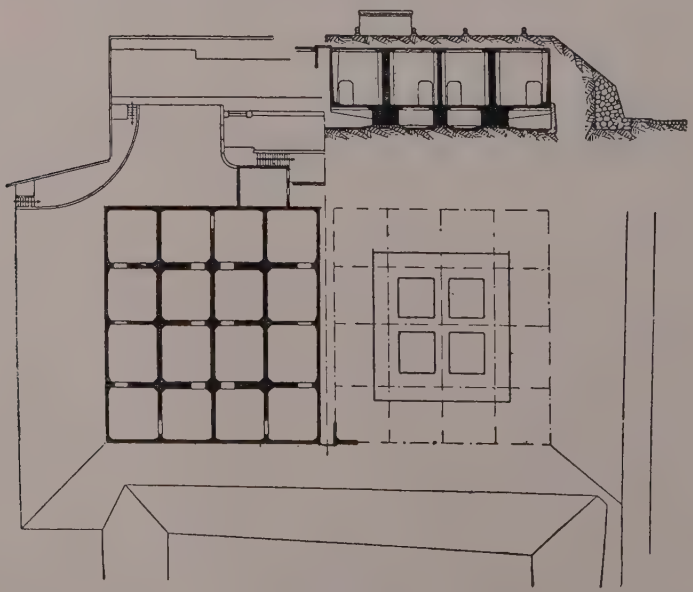


Fig. 24.3.

to replace a plain concrete reservoir which had been so badly damaged by subsidence due to mining that about one-half of the stored water escaped in 24 hours. No method of stopping the leaks proved successful. Calculations showed that a total settlement of 60 in. and a horizontal movement of 10 in. were to be expected as the workings progressed, and the diagram of settlement confirmed these figures for the three years after the completion of the structure. As it is undesirable to work to high stresses in designing a reservoir which is likely to settle, the dimensions of the various parts are unusually large. The arrangement of cross walls—acting as beams—and their extended footings are shown in plan and section (*Fig.* 24.4)

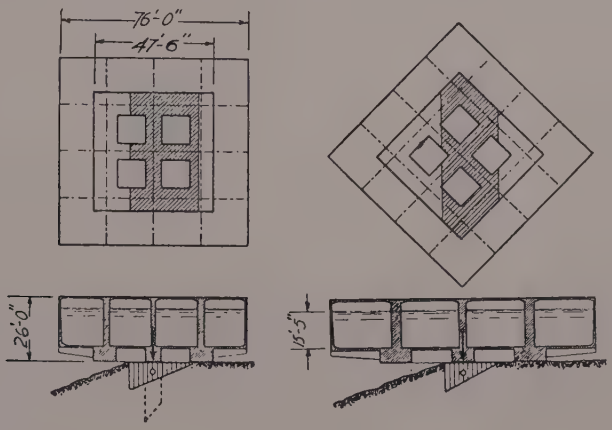


Fig. 24.4.

On this figure the assumed mode of support of the reservoir is shown for two cases, (1) bending in the plane of a centre-line of the reservoir, and (2) bending in the diagonal plane. Details of the arrangement of reinforcement in the walls are given in *Fig.* 24.5, which also shows the openings connecting adjacent cells. At midspan

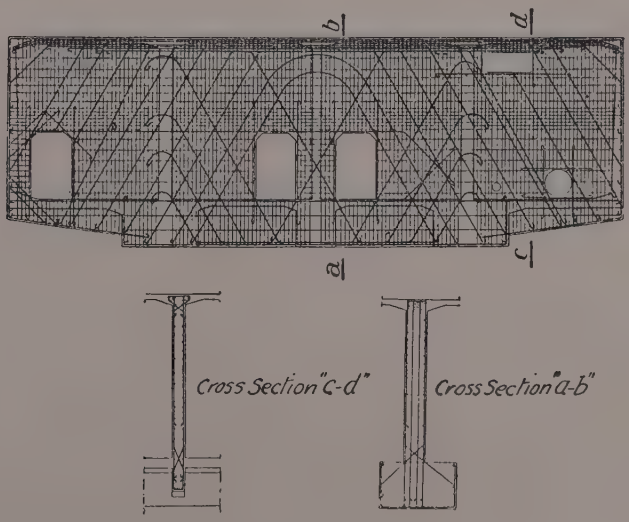


Fig. 24.5.

in the heavy walls there are twenty-eight 45-mm. (1·78-in.) bars in the top flanges, and the bending moment under the heaviest loading is 28,700,000 ft.-lb. Both floor and roof are designed as mesh-reinforced slabs.

The Redhill service reservoir (*Fig.* 24.6) of the Corporation of Nottingham has a capacity of 4,000,000 gallons and is designed to conform to ground movements and to reduce the stresses due to severe mining subsidence. This is necessary because of the existence of unworked seams of coal below the site. The average depth below the surface and the thickness of the four seams are: (1) 335 yd. deep, 4 ft. 4 in. thick; (2) 445 yd. deep, 3 ft. 3 in. thick; (3) 460 yd. deep, 2 ft. 5 in. thick; (4) 537 yd. deep, 3 ft. 6 in. thick. From the point at which mining operations were expected first to cause movement under the reservoir, the length of the coal-face was to be extended to the limits of the area of disturbance on each side of the reservoir, and mining continued below and beyond the reservoir to the point at which disturbance ceased. The rate of cutting in the area was to be about 40 yd. per month. From observations it was expected that the vertical settlement of the ground at the surface would be about 3 ft., that there would be considerable lateral pull, and that the mean gradient of differential settlement would be about 1 in 180.

The surface movements were such that a structure initially bearing continuously

Fig. 24.6.

on the ground might be subjected alternately to sagging and hogging effects. Therefore, for a reservoir of known depth, the greatest size of an economical structural unit which could safely resist the resulting stresses could be determined. It might appear that storage of the required capacity could be provided by a number of smaller reservoirs, each of the economical size determined, and the pipes and valves arranged to permit operation either individually or together. Investigation in this case, however, showed that a single reservoir of the design described in the following was much less costly and could be accommodated on a smaller area than several smaller reservoirs, and there were also operational advantages.

The depth of the reservoir is about 16 ft., and the mean depth of water 13 ft. The internal dimensions on plan are 230 ft. by 220 ft. For convenience of operation, the reservoir is divided by longitudinal and transverse walls into four equal compartments. The reservoir is also divided into twenty-five independent structural units, each about 46 ft. by 44 ft. Each rectangular unit comprises part of the floor and roof, and is bound along its sides either by lattice girders or walls which are designed as deep beams to resist ground movements as well as water pressure, dead weight, and the load on the roof. The units are connected only at floor level, where hinges are provided about which each unit can rotate. There is a gap between adjacent units to permit freedom of movement between the walls and roof. All pipes within the reservoir have flexible joints capable of accommodating the greatest

expected movement. In *Fig.* 24.7 are shown details (C) of the free joint in the roof, a horizontal section (B–B) through a watertight joint in the wall, and details (A) of a hinged joint in the floor. The hinged joints in the floor provide an anchorage between the units to enable the structure as a whole to resist lateral pull. The gaps in the walls are 4 in. wide when constructed and are made watertight by cover-plates of sheet lead weighing 10 lb. per square foot. At each joint in the wall there is an external inspection chamber as shown on the sectional plan in *Fig.* 24.7. A ground drain is provided around the reservoir immediately outside the walls. The gaps in the roof are covered with precast concrete slabs, bedded and jointed in a plastic composition.

In view of the high bearing value of the ground at foundation level, a 10-in. bed of clay puddle was provided over the whole area of the excavation and covered with 4 in. of plain concrete to form a level slab upon which the structure is founded. The plastic bed of clay considerably reduces the stresses resulting from sagging and hogging movements, and limits the pull transmitted from the ground. It was assumed that the clay would not yield under a pressure of about 1 ton per square foot, but that when movement occurred it would be capable of offering effective resistance to pressures not exceeding 3 tons per sq. ft. If the reservoir were founded directly upon the strong underlying ground, which would not yield under a pressure of less than 10 tons per sq. ft., the assumed supporting area would have had to be greatly reduced and the effective spans of the units greatly increased. In a modern tank a thick layer of compressible synthetic material could be substituted for puddle clay.

The walls retaining water span from floor to roof and, in this respect, were assumed to be completely fixed at the base and freely supported at the top. They were also designed to act as deep beams of a length equal to that of the units. A hinged joint similar to that between the units is provided between the foundations of the walls and girders and the floor, thereby limiting the weight of water carried by the walls and girders.

The floor is of laminated construction, formed of three thin slabs capable of following the changing contour of the ground without serious cracking. The slabs were concreted in panels, the construction joints in each of the layers being staggered. Open joints $\frac{3}{4}$-in. wide in the top slab are filled with plastic composition. The hinges in the floor are formed by gaps 9-in. wide between the foundations of the walls and girders and the lowest of the three layers of the floor slab. Bars of $\frac{1}{2}$-in. diameter cross the gaps horizontally and are embedded in the concrete on both sides at the level of the centre of the lowest layer. The gaps are filled with a plastic composition on which the two upper layers of the floor slab were placed.

Within each unit there are four columns supporting the roof (*Figs.* 24.8 and 24.9). Each column is supported on a separate base, which is connected to the floor only by flexible joints of the same type as elsewhere.

I.C.E. Report.—The comments in the following are abstracted from a 'Report on Mining Subsidence' published by the Institution of Civil Engineers (1959).

Most older reservoirs and storage tanks are constructed, according to the practice of the time, of mass concrete or brickwork with puddled clay under the floors and behind the walls. This form of construction has proved generally effective in counteracting the effects of movement but this may be due to the fact that most of the reservoirs and tanks are relatively small. This method of construction still has its advocates, but a monolithic structure of reinforced concrete with columns and with either a beam-and-slab or flat-slab roof is now more general. Expansion

and contraction joints are normally not provided and the roof and floor are continuous with the walls. Small temperature cracks are sealed by ironing-in bitumen.

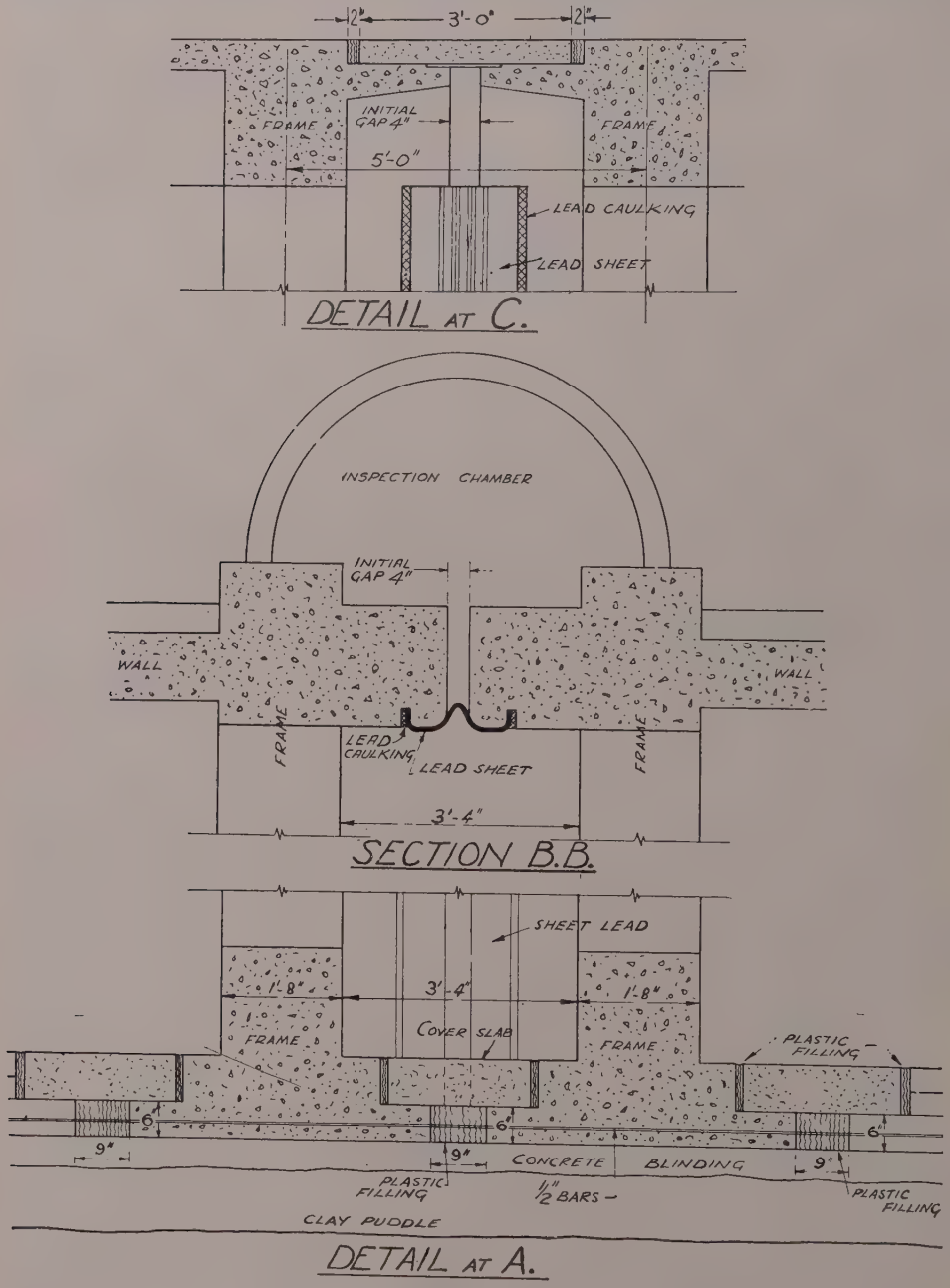


Fig. 24.7.

Copper or rubber seals are normally employed to ensure watertightness at the construction joints of the walls. Variations of this practice are: (*a*) to provide

freedom of movement between the walls, roof and floor; (*b*) to ensure in the design and construction that cracks due to subsidence will occur along pre-determined planes.

Fig. 24.8.

Fig. 24.9.

In general, reservoirs and tanks should be small, that is in units of from 250,000 to 500,000 gallons capacity, the units being inter-connected, if necessary, by flexible-joint piping. For larger capacities, however, one reservoir with independent sub-divisions is often preferred.

It is impossible, since it would prove uneconomical, to design for the stresses which are likely to arise from subsidence because these are indeterminate and, subject to the foregoing, it is unnecessary to depart from the requirements of the British Standard Code of Practice for reinforced and prestressed concrete structures containing liquids (CP.2007:1960. Appendix I).

In the author's opinion, this report is both self-contradictory and defeatist.

Prestressed Concrete Tanks

If and when it is possible to produce high strength steel with an elongation of more than 10 per cent., the use of prestressed concrete, utilizing the floor and roof of the reservoir as I-beam flanges, may be considered.

In some existing post-tensioned prestressed structures the bars or cables are set in small individual ducts which are then pressure-grouted. The author has always regarded this practice as unsound in water-retaining structures. Details of the collapse of prestressed work may be found in 'Engineering News Record' for May 4th 1961 and January 4th 1962.

CHAPTER TWENTY-FIVE

CIRCULAR TANKS UNDER LATERAL LOADING

In this country any lateral loading is usually wind load.

When a large open-topped circular tank stands above ground level and is standing empty the wind will tend to push in the top of the windward section of the wall. If the tank is comparatively shallow and the wall is relatively thick, any horizontal force applied to the top of the wall may be transferred directly by local cantilever action to the floor of the tank. However, if the tank is comparatively deep and the walls relatively thin there will be considerable local bending of the top section of the wall in a horizontal plane which, in an extreme case, might lead to collapse of the tank. The writer knows of no such case occurring with reinforced concrete tanks but at least one large steel tank has failed in this way. In this country calculations of the type that follow will only be necessary where the tank wall carries a row of upstanding stanchions or some type of superstructure that greatly increases the effects of wind.

In *Fig.* 25.1 a plan and section of a tank subjected to wind loading is show

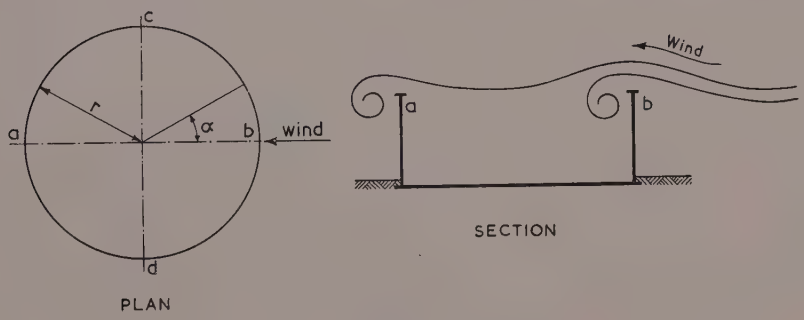


Fig. 25.1.

The wind effect is mostly due to discontinuity of the stream lines over the edge of the wall (turbulence) and it will be assumed that the effect varies as $\cos^2 \alpha$ and that three-quarters of the effect comes on the windward half of the rim and one-quarter on the leeward half. It will be assumed that the effect is symmetrical about diameter *ab* and that the top of the wall is provided with a stiffening ring to resist the local bending. To give a problem of some tangible size the total wind load will be assumed to be 10,000 lb. and this will be divided into a series of concentrated loads applied at 36 points, spaced at 10-deg. intervals, so that standard tabulated values can be used without the need to integrate. There are thus 18 loads (or rather 17 loads and 2 half-loads) on the semicircle *acb*, spaced at 10-deg. intervals as shown in *Fig.* 25.2.

The total horizontal wind load of 10,000 lb. that comes on the top stiffening beam is transferred to the ground by the wall of the tank acting altogether as a vertical cantilever of hollow circular section. Following the usual theorem for beam shear the shearing force is tangential to the circle and the intensity per unit length

of circumference is $\frac{10{,}000}{\pi r}$ sin α lb. per foot, where r is the radius in feet. Dividing the circumference of the semicircle into 18 parts and resolving the shearing forces

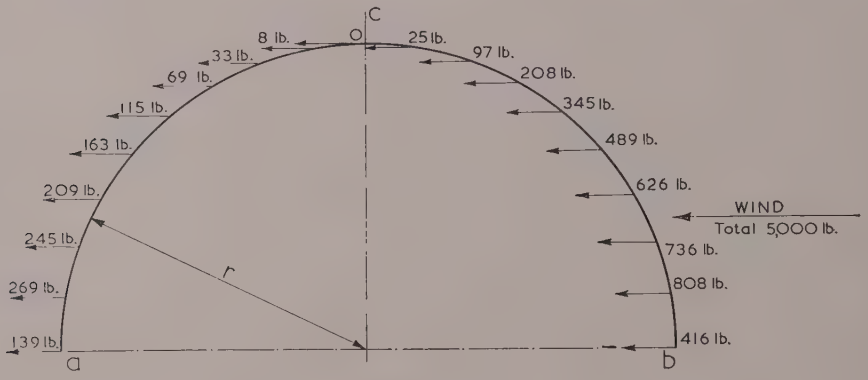


Fig. 25.2.

parallel to, and at right-angles to, the diameter *ab* the forces shown in *Fig.* 25.3 are obtained.

The top stiffening beam is held in equilibrium by the wind loads shown in *Fig.* 25.2 and the shearing forces shown in *Fig.* 25.3.

Symmetrical cases of this kind may be solved by first treating one-half of the

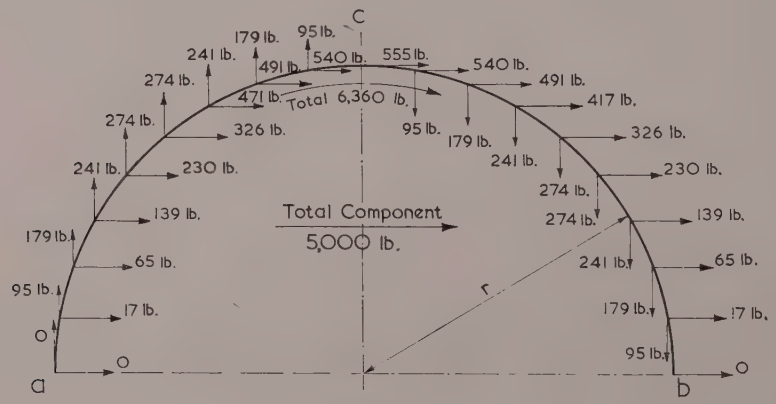


Fig. 25.3.

ring beam (that is, the semicircle *acb*) as a fixed-ended semicircular arch and then applying a correction, as there can be no ' horizontal ' thrust (that is, no shear in the ring beam) at *a* and at *b*. The analysis, the coefficients and the ' temperature ' correction factors may be found in the writer's ' Reinforced Concrete Design '.

The effect of the shearing stresses shown in *Fig.* 25.3, with a total resolved component of 5,000 lb., on a fixed-ended semicircular arch of radius r is shown in *Fig.* 25.4. If, instead of splitting the shearing force into separate concentrated loads, the problem is integrated directly, $H_b = 2{,}500$ lb., $T_b = 2{,}388$ lb. and $M_b = + 795r$. Positive moments cause tension on the inside of the circle. These are the same for any symmetrical distribution of the wind force of 5,000 lb. but are correct only

within the limits of slide-rule calculation. The effect of the wind loads shown in *Fig.* 25.2 on a fixed-ended semicircular arch, is shown in *Fig.* 25.5 These figures apply only to the exact distribution of wind loads shown in *Fig.* 25.2.

Combining *Figs.* 25.4 and 25.5 the end reactions in *Fig.* 25.6 are obtained. This

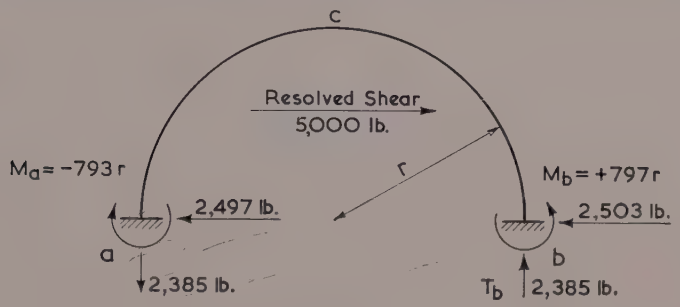


Fig. 25.4.

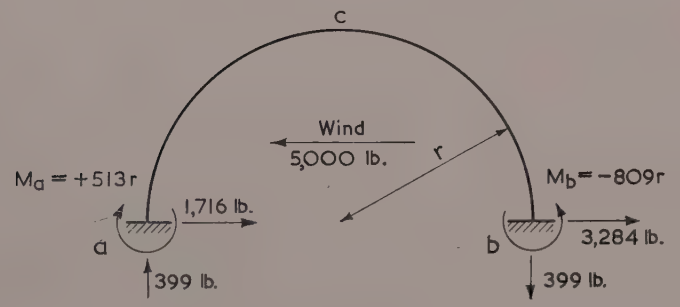
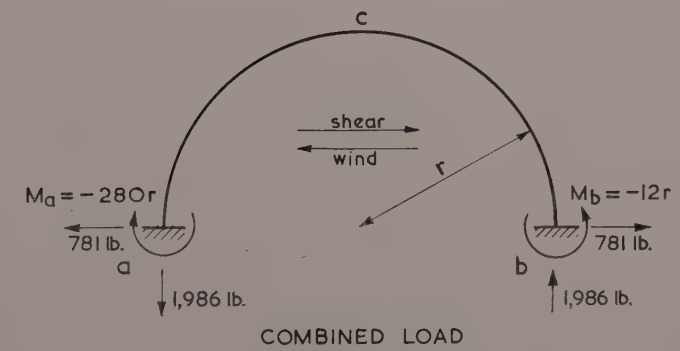


Fig. 25.5.



COMBINED LOAD

Fig. 25.6.

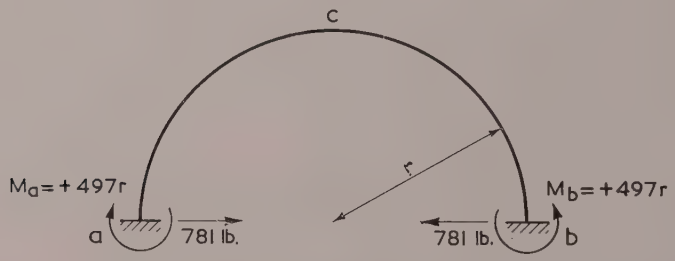
figure shows an outward thrust of 781 lb. at *a* and at *b*. But, since the loading is symmetrical about the main centre-line of the ring beam *ab*, there can be no shear reaction at *a* and *b* at right-angles to the ring, and points *a* and *b* will move inwards towards one another until this movement produces an inward thrust of

781 lb. to cancel out the outward thrust of 781 lb. in *Fig.* 25.6. Using the temperature coefficients:

$$H = \frac{6{\cdot}74\phi REI}{r^2} = 781, \text{ giving } \frac{\phi REI}{r} = \frac{781r}{6{\cdot}74}.$$

$$M_b = 4{\cdot}28\,\frac{\phi REI}{r} = \frac{4{\cdot}28}{6{\cdot}74} \times 781r = 497r.$$

This correction is shown in *Fig.* 25.7. Applying the correction to the values in



CORRECTION DUE TO SHORTENING OF a b.

Fig. 25.7.

Fig. 25.6, the fixed-ended arch is transformed into a continuous circular ring-beam with thrusts and moments at *a* and *b* as shown in *Fig.* 25.8. From these and the

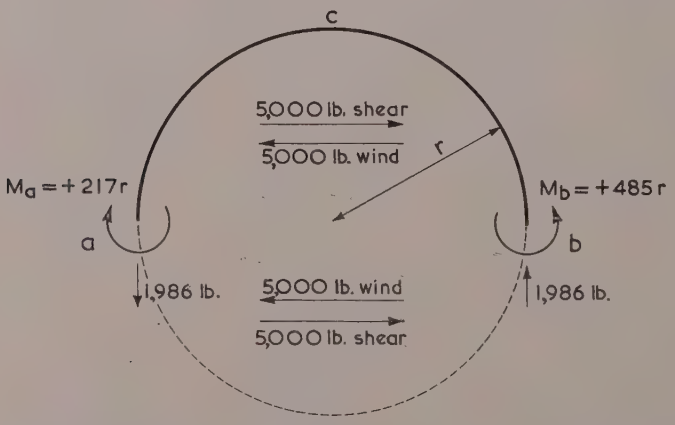


Fig. 25.8.

detailed loads in *Figs.* 25.2 and 25.3 the moments, thrusts and shears may be calculated at all points along the ring. The values of the thrusts and shears are small. The moments in the more heavily-loaded quadrant are as shown in *Fig.* 25.9. If the wind loads in *Fig.* 25.2 are doubled the thrusts and moments in *Figs.* 25.8 and 25.9 will be doubled. For example, if the total effective wind load on the tank is 10 tons and $r = 100$ ft.,

$$M_b = \frac{22{,}400}{10.000} \times 485 \times 100 = 108{,}900 \text{ lb.-ft.} = 1{,}300{,}000 \text{ lb.-in.}$$

If the top of the wall is not less than 20 in. in thickness then the top 3 feet of the wall itself could be reinforced to resist this moment and no additional ring beam would be needed (unless required to form a walkway).

If the tank wall carries a superstructure and if the designer has to comply with

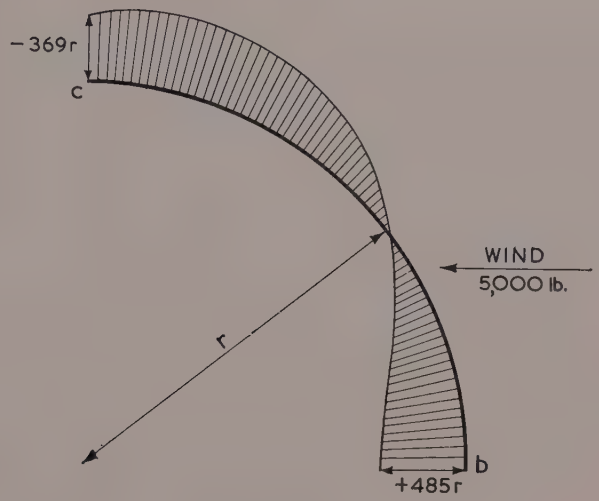


Fig. 25.9.

some arbitrary clause in local regulations or in some code of practice by which the total wind load must be assumed to be distributed uniformly over the elevation, then the loading will be as shown in *Fig.* 25.10. Splitting this into a series of concentrated loads spaced at intervals of 10 degrees, the distribution shown in the figure is obtained.

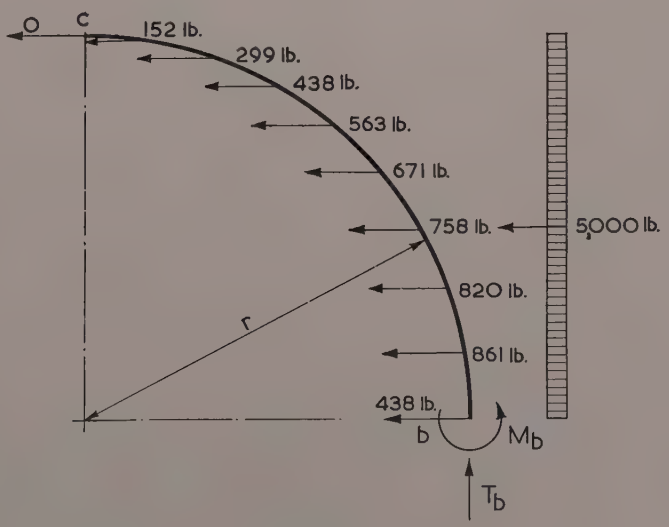


Fig. 25.10.

Applying the same process of analysis that was used for the loading in *Fig.* 25.2,

$$M_b = + 660r \quad \text{and} \quad T_b = + 1{,}856 \text{ lb.}$$

Integration of this particular case gives (within the limits of slide-rule accuracy),

$$M_b = + 0{\cdot}140pr^2, \; M_c = - 0{\cdot}125pr^2 \text{ and } M_a = + 0{\cdot}110pr^2,$$

positive moments producing tension on the inside of the circle.

In *Fig.* 25.10, $p = 5{,}000/r$ and $M_b = + 0{\cdot}14 \times \dfrac{5{,}000}{r} \times r^2 = + 700r$,

against a figure of $+660r$ previously calculated. The discrepancy of less than 6 per cent. is mostly due to cumulative errors in the use of a slide-rule.

If a tank with a diameter of 200 ft. carries a superstructure 30 ft. high which is

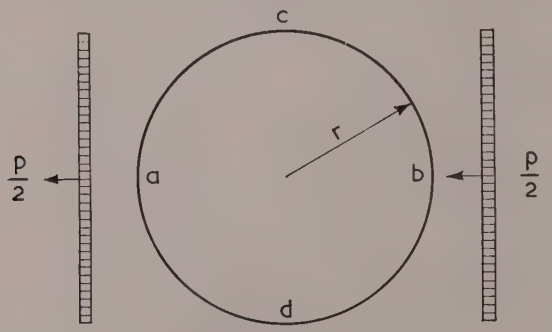


Fig. 25.11.

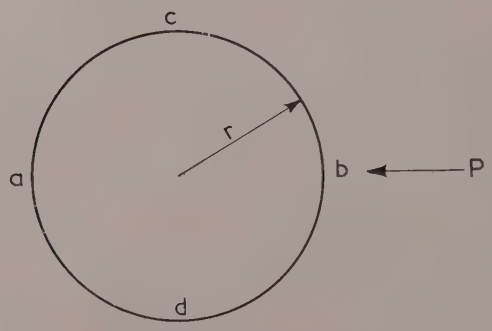


Fig. 25.12.

subject to a wind load of 18 lb. per sq. ft., and if all the wind load is to be carried on the windward columns and is spread uniformly across the elevation as shown in *Fig.* 25.10:

$$\text{Total wind load} = 200 \times 30 \times 18 = 108{,}000 \text{ lb.},$$

against the assumed figure of 10,000 lb. in *Fig.* 25.10, and $r = 100$ ft.

$$M_b = \frac{108{,}000}{10{,}000} \times 660 \times 100 = 712{,}000 \text{ lb.-ft.} = 8{,}550{,}000 \text{ lb.-in.}$$

To resist this with a resistance moment of $100\ bd^2$, the top 3 feet of wall would need to be 4 ft. in thickness.

If the uniform pressure p on the windward face in *Fig.* 25.10 were split to give a uniform pressure of $\frac{1}{2}p$ on the windward face and a uniform suction of $\frac{1}{2}p$ on the leeward face, as shown in *Fig.* 25.11, then $M_b = + 0{\cdot}015pr^2$, $M_c = 0$ and $M_a = - 0{\cdot}015\ pr^2$.

If the wind loading were concentrated into a single concentrated load P as in *Fig.* 25.12,

$$M_b = + 0{\cdot}238Pr.$$

Wind on Buried Tanks

So far it has been assumed that the tank is standing well above ground level, but most tanks subject to heavy horizontal loading are surrounded by substantial earth banks so that the wall of the tank may move inwards but cannot move outwards against the counterpressure of the earth.

The illustration in *Fig.* 25.13 represents a tank ACBD inside earth banking subject

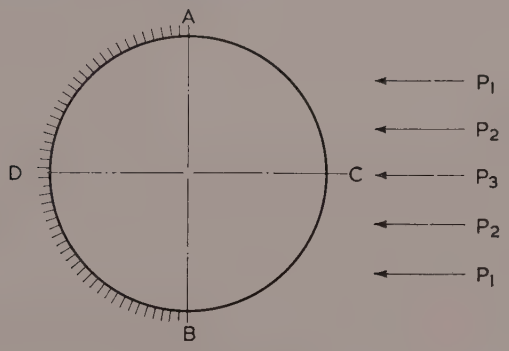


Fig. 25.13.

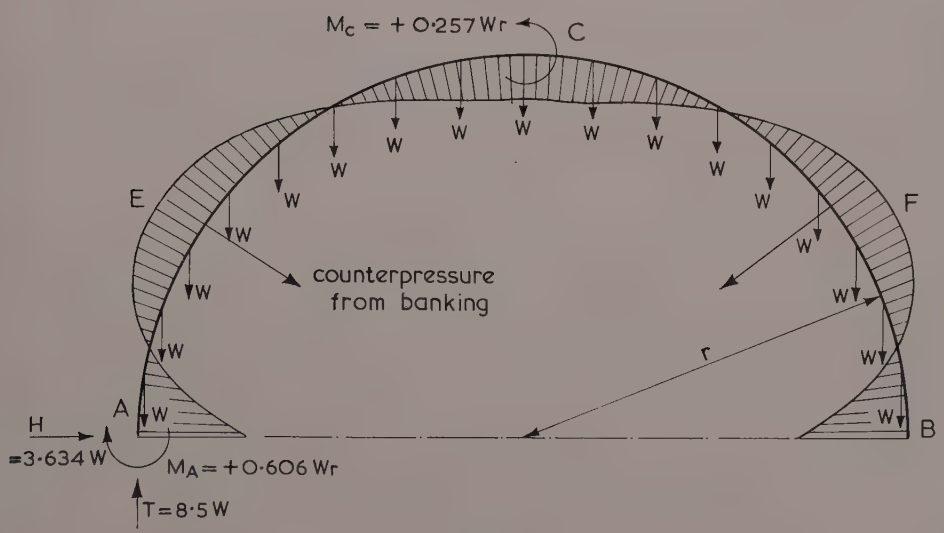
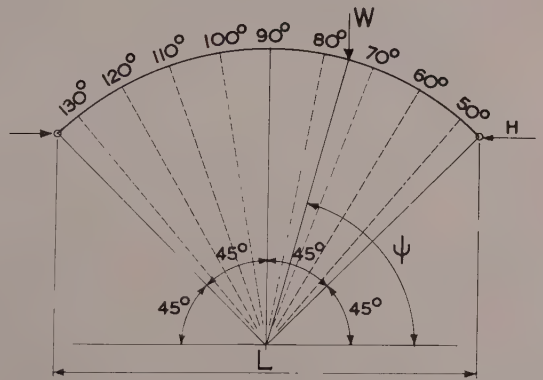


Fig. 25.14.

to a series of lateral forces $P_1P_2P_3$ applied to the top of the wall. If the earth has been carefully placed and consolidated the leeward half of the tank ADB may be assumed to be continuously supported by the bank so that the top of the windward half of the wall ACB acts primarily as a semicircular fixed-ended arch. The moments in this semicircular arch would be of the type shown in *Fig.* 25.14, but



TWO HINGED CIRCULAR ARC OF 90°.

E. I. CONSTANT.

For Temperature Rise

$$H = \frac{40\phi REI}{L^2}$$

where ϕ = coefficient of thermal expansion per 1 degree

R = rise in temperature in degrees

LOAD W ON POINT	MOMENT IN TERMS OF WL AT POINTS								
	130°	120°	110°	100°	90°	80°	70°	60°	50°
ψ = 50°	−0.0031	−0.0072	−0.0086	−0.0071	−0.0028	+0.0040	+0.0132	+0.0247	+0.0380
ψ = 60°	−0.0106	−0.0248	−0.0303	−0.0261	−0.0124	+0.0101	+0.0405	+0.0785	+0.0213
ψ = 70°	−0.0161	−0.0368	−0.0431	−0.0335	−0.0087	+0.0305	+0.0820	+0.0340	+0.0049
ψ = 80°	−0.0175	−0.0377	−0.0393	−0.0205	+0.0170	+0.0720	+0.0241	−0.0017	−0.0064
ψ = 90°	−0.0152	−0.0290	−0.0205	+0.0101	+0.0618	+0.0101	−0.0205	−0.0290	−0.0152

ψ	H
50°	0.123W
55°	0.279W
60°	0.413W
65°	0.539W
70°	0.665W
75°	0.770W
80°	0.828W
85°	0.886W
90°	0.908W

Fig. 25.15.

under these moments, although the crown of the arch at C moves away from the earth banking, the haunches at E and F would tend to move outwards but would be prevented by counterpressure from the earth and, if the bank were really solid, that part of the tank wall which is immediately facing the wind would be in some state similar to a two-hinged segmental arch subtending an arc of about 90 deg., as shown in *Fig.* 25.16.

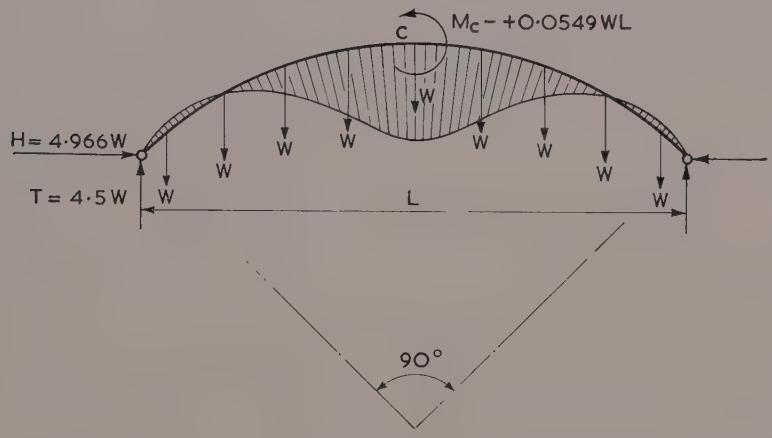


Fig. 25.16.

The values of H for various positions of an applied load W on a two-hinged segmental arch of 90 deg. are given in *Fig.* 25.15.

The reduction of the system to a segmental arch should only be assumed if the earth banking is of suitable material which has been systematically laid and consolidated. With poorer material the windward half may be treated as a semicircular fixed-ended arch, while with very poor material or when the banking may be removed at some future date, the top of the tank wall should be designed as a free-standing tank using the methods described earlier in this chapter.

Tanks Subject to Seismic Effects

The present-day method of considering the effects of lateral earthquake forces is to assume that the force is equivalent to the application of a static horizontal load which represents some percentage of the weight of the structure. The values in *Figs.* 25.14 and 25.16 are actually taken from the design of a circular wall carrying vertical loading and subject to a horizontal earthquake surge equal to 0·1225 *g*. In *Figs.* 25.14 and 25.16 the north half of the wall is shown with the earthquake surging from north to south. In addition to the surge of the wall and superstructure there will be wave-slap against the south half of the wall. According to expert opinion this effect is small and it merely forces the wall against the banking.

Wind Effects on a Tank for a Spirally-guided Gasholder

The term 'spiral' is used (or rather terminologically mis-used) in the same sense as in a 'spiral' staircase, the guides being attached to the holder in a series of regular helices at an angle of about 45 deg. and each guide running between two flanged rollers which are fixed to the top of the wall of the gasholder tank (*Fig.* 25.17).

Usually the makers of the gasholder will give details of the tangential and radial forces on the pairs of rollers that keep the holder in position. The reader might like to check these for himself.

The manner in which the lateral wind load on the holder is transmitted to the top of the tank wall must depend on the stiffness of the holder (which is stiffened by internal gas pressure), on the clearances between the guides and the roller, and on the accuracy with which the guides and rollers are set.

When not subjected to wind loads the holder is stable and when it heels over slightly it tends to right, in the same manner as a ship. The amount of this tendency

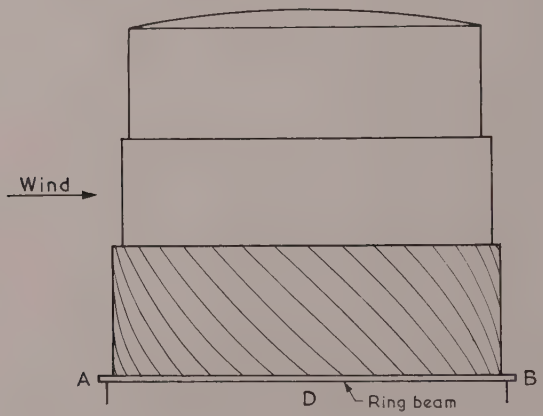


Fig. 25.17.

depends on the ratio between the height and the diameter and on the relative weights per square foot of the roof, and the upper, middle and lower parts of the wall. The natural tendency against capsizing is small and will be ignored in the calculations that follow.

The holder, stiffened by internal pressure, will act as a vertical cantilever and will impose both a bending moment and a horizontal shearing force on the top of the tank wall. The stresses due to the moment about a horizontal axis will be primarily vertical compression and tension but, due to the inclination of the guides, horizontal forces will develop and these may tend to distort the stiffening ring at the top of the tank wall. For simplicity the moments and shearing forces may be considered separately.

Forces due to Overturning Moment

The tank will have a number of pairs of rollers set at equal intervals round the circumference; for ease in calculation a continuous narrow ring of thickness t (*Fig.* 25.18) may be considered instead. The modulus Z of a thin ring about a horizontal axis is $\pi r^2 t$. If the wind moment about the axis CD is M, the primary vertical stress at any point P in *Fig.* 25.18 is $\frac{M}{\pi r^2 t} \cos \theta$. Assuming that the guides are set at an angle of 45 deg., this vertical stress induces a stress of $\frac{\sqrt{2}M}{\pi r^2 t} \cos \theta$

on the face of the roller and thence a horizontal stress tangential to the circle equal to $\frac{M}{\pi r^2 t}\cos\theta$ (*Fig.* 25.19).

The total moment of this tangential stress about the vertical axis of the circle for the semicircle CBD is

$$2\int_0^{\frac{1}{2}\pi}\frac{M}{\pi r^2 t}\cos\theta \times r \times r.d\theta \times t = \frac{2M}{\pi}.$$

If the guides are set in a left-hand screw as in *Fig.* 25.17, the tangential forces on the

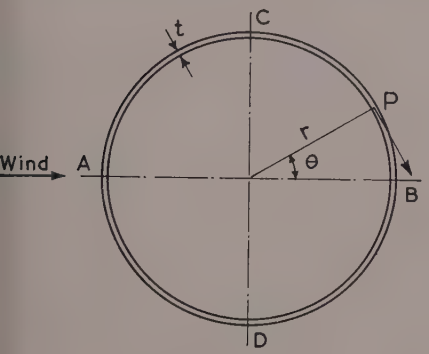


Fig. 25.18.

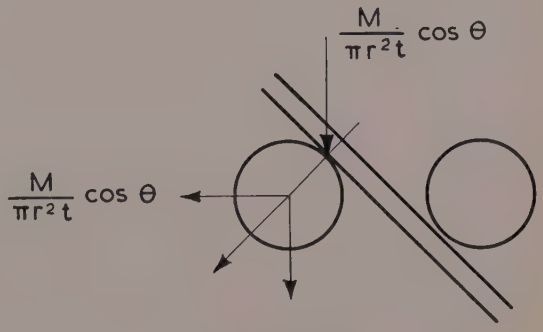


Fig. 25.19.

semicircle CBD will all run from C towards D and those on the semicircle CAD will also run from C towards D. Each will have a component acting parallel to CD. On a small length of ring *ds* at P, the component will be $\frac{M}{\pi r^2 t}\cos^2\theta(r.d\theta)t$ and the total component in this direction over the whole circle will be

$$4\int_0^{\frac{1}{2}\pi}\frac{M}{\pi r}\cos^2\theta.d\theta = \frac{M}{r}.$$

In other words an overturning moment M about the horizontal axis CD tends to drive the holder in the direction DC with a force equal to $\frac{M}{r}$. Assuming that this movement is taken by the guides bearing on the flanges of the rollers in radial bearing and that the intensity varies as sin θ, being zero at A and B, equal to $f_{max.}$ at C and D, and generally equal to $f_{max.}\sin\theta$ (see *Fig.* 25.20), the component parallel to CD of the total stress on an element *ds* at point P is

$$f_{max.}\sin\theta\,(r.d\theta)t\sin\theta$$

and the total component over the whole circle is

$$4\int_0^{\frac{1}{2}\pi} f_{max.}rt\sin^2\theta\,d\theta = f_{max.}\pi rt.$$

This must equal $\frac{M}{r}$ so that $f_{max.} = \frac{M}{\pi r^2 t}$.

To obtain some tangible values, assume that there are 36 pairs of rollers, each serving an arc of 10 deg., and that $M = 80r^3$. When $\theta = 45$ deg.,

$$\text{tangential stress at P in } \textit{Fig.}\ 25.18 = \frac{M}{\pi r^2 t}\cos\theta = \frac{M}{\pi r^2 t}\times\frac{1}{\sqrt{2}}.$$

Area of portion of tank surface given by an arc of 10 deg. $= \frac{\pi rt}{18}$.

Total tangential force on arc $= \frac{M}{\pi r^2 t} \times \frac{1}{\sqrt{2}} \times \frac{\pi rt}{18} = \frac{M}{18\sqrt{2}r} = \frac{80r^3}{18\sqrt{2}r} = 3{\cdot}14r^2$.

The radial and tangential forces on the ring beam due to eight 10-deg. arcs of wall

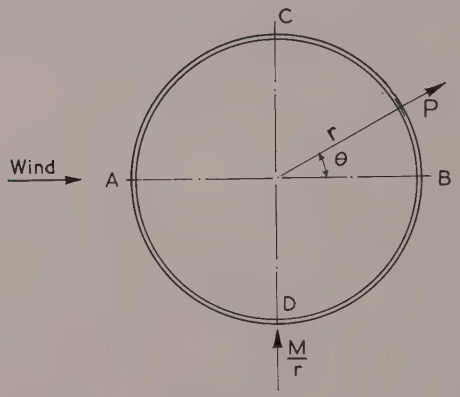


Fig. 25.20.

subjected to a moment of $80r^3$ (always assuming the guides are set at 45 deg.) are shown in *Fig.* 25.21. For other values of M and with a different number of

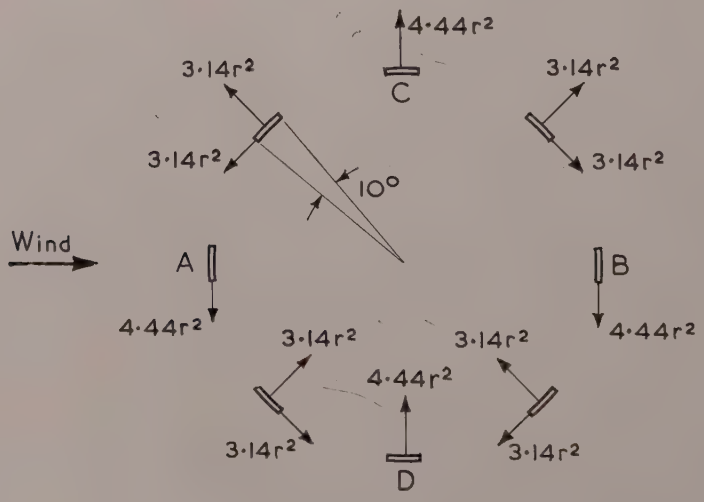


Fig. 25.21.

rollers, the forces may be found pro-rata. Thus if $M = 56r^3$ and there are 48 pairs of rollers, the value of $3{\cdot}14r^2$ per pair of rollers becomes $3{\cdot}14r^2 \times \frac{56}{80} \times \frac{36}{48} = 1{\cdot}65r^2$. In order to use standard tables 36 arcs of 10 deg. are considered here and these forces must be resolved parallel to AB and CD.

In general, the force on an arc on 10 deg. at an angle θ is

$$\frac{M}{18r}\cos\theta \text{ tangentially} \quad \text{and} \quad \frac{M}{18r}\sin\theta \text{ radially.}$$

The resultant of these forces parallel to AB is

$$\frac{M}{18r}\cos\theta\sin\theta + \frac{M}{18r}\cos\theta\sin\theta = \frac{M}{18r}\sin 2\theta.$$

The resultant parallel to CD is $\frac{M}{18r}\cos^2\theta - \frac{M}{18r}\sin^2\theta = \frac{M}{18r}\cos 2\theta.$

The components of the horizontal forces due to a moment of $80r^3$ for the semi-circle CAD are shown in *Fig.* 25.22, the loading being symmetrical about CD.

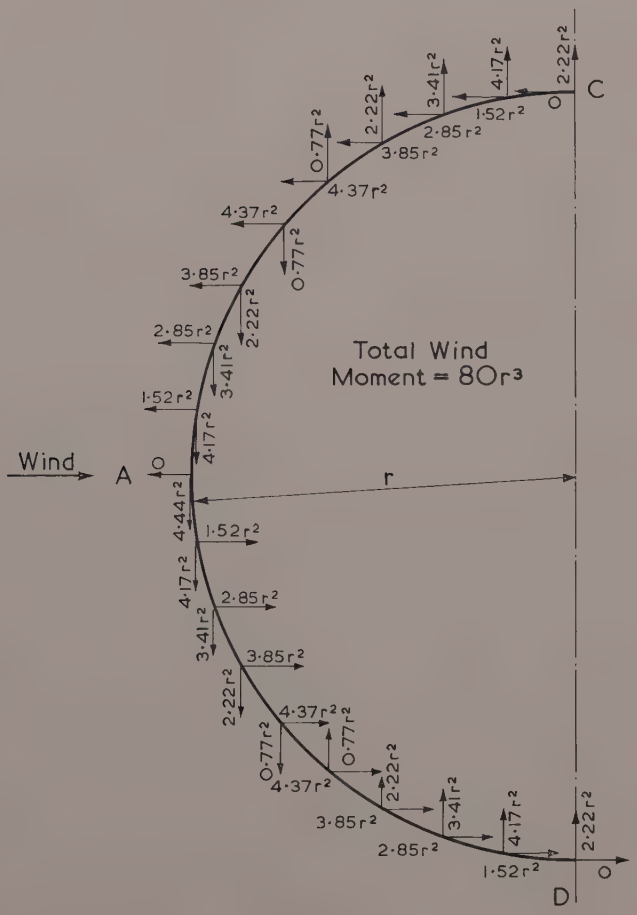


Fig. 25.22.

Firstly, it will be seen that the horizontal loading from the wind moment on the ring beam as a whole is in equilibrium (as it may be expected to be) and, even if the ring beam were not attached to the rest of the wall, it would not move bodily. Any tangential shearing forces that may develop between the wall and the ring beam must be very small and may be ignored.

Treating the semicircle CBD as a semicircular fixed-ended arch the end reactions shown in *Fig.* 25.23 are obtained. Since there is no shear at C and D no ' tempera-

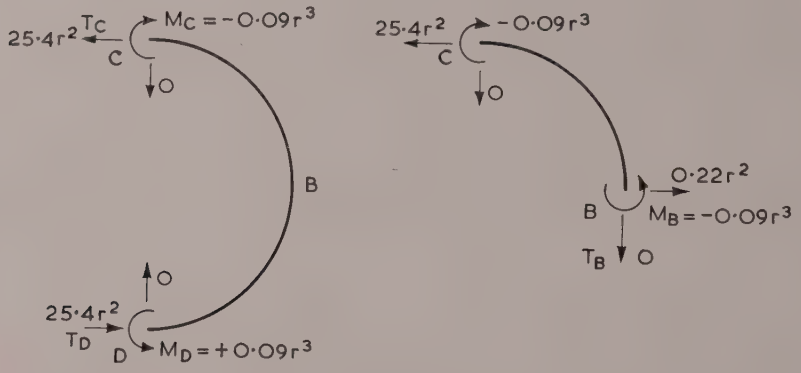


Fig. 25.23.

ture ' correction (as described in an earlier example) is required. These figures are only correct within the limits of the slide-rule.

Taking moments about the vertical axis of the circle for the quadrant CB and resolving in both directions, the reactions at B shown in *Fig.* 25.23 are obtained.

Forces due to Shear

If the wind creates a horizontal shear force F in the direction A towards B, this force can be imagined to be primarily distributed uniformly between all the guides. In this case the guides will bear on the flanges of the rollers at A and B and on the faces of the rollers at C and D. But any horizontal force on the face of a roller causes a vertical force of the same magnitude (still assuming that the guides are set at 45 deg. and ignoring friction). This force would drive the holder down at D and up at C (still assuming that the guides are set in a left-handed screw). Apart from the natural stability of the holder, which it has already been decided to ignore, there is nothing to resist these vertical forces and the holder would tilt about the axis AB and move from A towards B until all the shearing force comes on the flanges of the rollers in a series of radial forces (*Fig.* 25.20). Assuming that the intensity of each radial reaction varies as $\cos\theta$, the component of the radial force in the direction of the wind is $f_{max.} \cos\theta\ (r.d\theta)t \cos\theta$.

Total shear $= F = 4\int_0^{\frac{1}{2}\pi} f_{max.} rt \cos^2\theta\, .d\theta$, or $F = f_{max.}\pi rt$.

Thus $f_{max.} = \dfrac{F}{\pi rt}$.

To give the problem some tangible size, assume $F =$ 10,000 lb. With 36 pairs of guides spaced at 10-deg. intervals, the total wind shear of 5,000 lb. on the semicircle ACB would produce the forces shown in *Fig.* 25.24. Treating ACB as a semicircular fixed-ended arch, under this loading the end reactions at B will be as shown in the figure. Since the applied loads in *Fig.* 25.24 do not balance, they must be resisted by beam shears from the wall acting on the underside of the ring beam. These have already been determined from a previous solution and are

shown in *Fig.* 25.3. Combining *Figs.* 25.3 and 25.24, the end reactions at A and B are, within the limits of slide-rule accuracy, as shown in *Fig.* 25.25. Summing the forces on the quadrant CB the reactions at C may be obtained.

Considering a gasholder 200 ft. in diameter and 180 ft. high, and assuming an

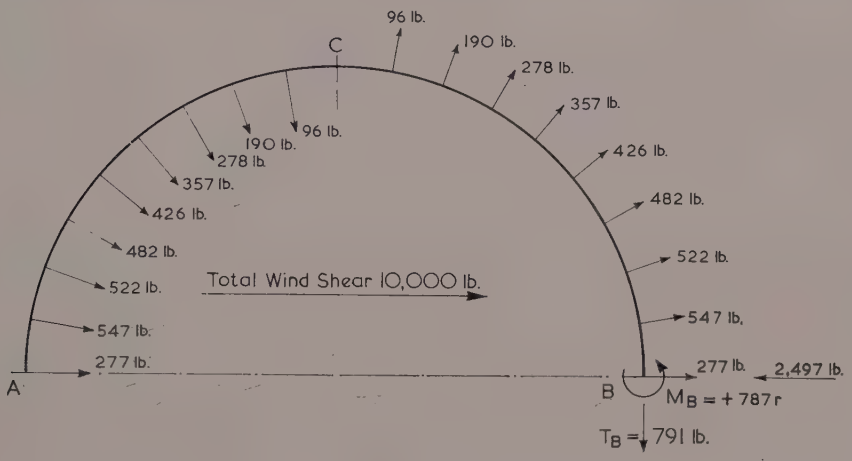


Fig. 25.24.

average wind pressure of 18 lb. per sq. ft. on the elevation with the centre of pressure 100 ft. above the top of the tank,

$F = 18 \times 200 \times 180 = 684{,}000$ lb. and $M = 648{,}000 \times 100 = 64{,}800{,}000$ lb.-ft.

As $r = 100$ ft., $M = 64{\cdot}8r^3$ against the value of $80r^3$ previously assumed. The moments, thrusts and shears in the ring beam due to the overturning moment of $64{\cdot}8r^3$ will be 0·81 times those in *Fig.* 25.23, and the moments, thrust and shears in the ring beam due to the wind shear will be 64·8 times those shown in *Fig.* 25.25.

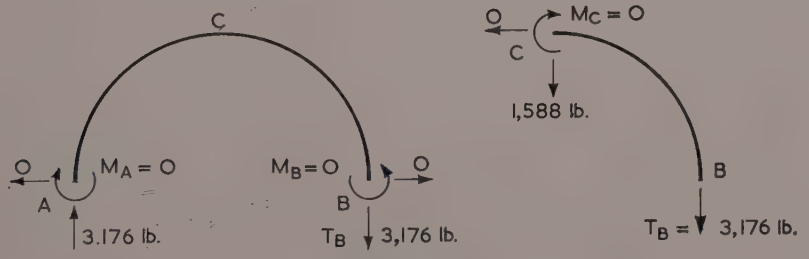


Fig. 25.25.

Combining these two sets of values,

$M_B = -73{,}000$ lb.-ft. $+ 0$; $T_B = 0 + 206{,}000$ lb. (tension).

Shear at B $= 1{,}780$ lb. $+ 0$.

$M_C = -73{,}000$ lb.-ft. $+ 0$; $T_C = 206{,}000$ lb. $+ 0$ (tension).

Shear at C $= 0 + 103{,}000$ lb.

$M_D = +73{,}000$ lb.-ft. $+ 0$; $T_D = 206{,}000$ lb. $+ 0$ (compression).

Shear at D $= 0 + 103{,}000$ lb.

Moments, thrusts and shears at intermediate points may be calculated by simple arithmetic.

These figures have been determined by the use of a slide-rule and are based on certain assumed distributions of loading. These assumptions depend on the relative stiffness of the holder and the clearances and tolerances, both radial and circumferential, between guides and rollers. Nothing has been allowed for slight inaccuracies in the guides or rollers. If the tank supported a column-guided holder of the same size, and if the columns were so braced as to spread the wind load evenly over both elevations this would give

$$\frac{648{,}000}{2 \times 200} = 1{,}620 \text{ lb. per foot run of diameter.}$$

$$M_B = 0{\cdot}03 \times 1{,}620 \times 100^2 = 486{,}000 \text{ lb.-ft.}$$

If the columns in the ring are so braced that all the wind load comes on the one elevation,

$$M_B = 0{\cdot}14 \times 3{,}240 \times 100^2 = 4{,}530{,}000 \text{ lb.-ft.}$$

The reader should study these comparative figures before finally deciding on the design of the ring beam.

Wind loading can only come on the top of the tank wall when the tank is full of water and the water pressure will tend to stiffen the tank.

Thrusts on Rollers

From *Fig.* 25.21, the radial outward thrust on the roller half-way between C and B is $3{\cdot}14r^2$ for a wind moment of $80r^3$, assuming the rollers are set at 10-deg. intervals. If the moment is $64{\cdot}8r^3$ and if there are actually 48 pairs of rollers, the radial thrust per pair of rollers is $\frac{64{\cdot}8}{80} \times \frac{36}{48} \times 3{\cdot}14r^2 = 1{\cdot}91r^2$.

If $r = 100$ ft., the radial thrust per pair of rollers is 19,100 lb.

From *Fig.* 25.24 the radial thrust on this pair of rollers is 393 lb. for a shearing force of 10,000 lb. and with 36 pairs of rollers. For a wind shear of 648,000 lb. on 48 pairs of rollers, the thrust would be $\frac{648{,}000}{10{,}000} \times \frac{36}{48} = 19{,}100$ lb.

Total thrust $= 19{,}100 + 19{,}100 = 38{,}200$ lb. $= 17$ tons.

Wind Effect on Large Structures

Circular storms varying in size from ' dust devils ' or ' twisters' to typhoons are often encountered at ground level in hot climates. Air turbulence of this type often develops in temperate climates and is frequently encountered by aeroplanes (occasionally with fatal results) but the effect seldom reaches down to ground level. When it does it produces destruction over a narrow belt which is of limited length. Any large structure which happens to stand directly in the path of the disturbance may be subjected to heavy pressure on half its elevation and heavy suction over the other half, and the structure may suffer some form of spiral collapse. The Hamman dome at the 1950 Toronto Fair had special bracing provided solely to guard against this effect. The higher the structure the more likely it is to reach up into the main area of turbulence, and a severe circular storm striking a large gasholder would probably wreck the upper lifts of the holder before the stresses in the gasholder

tank reached failure point. The author has had experience of only two such circular storms in this country and the chances of any given structure being struck by one are fairly remote. However, it is possible, when a high structure is surrounded by other high structures, for a strong steady wind to be deflected and broken up into a state of turbulence resembling a circular storm.

There are at least three recorded instances (one on film) of structures being struck by periodic gusts that coincided with the natural period of the structure. These conditions are unlikely to develop in a reinforced concrete structure itself but could develop in a metal superstructure carried on a reinforced concrete substructure.

Gasholder Tanks

The principles of the design of gasholder tanks are similar to those given in Chapters 2, 3 and 5, but the strictest precautions must be taken to prevent unequal settlement, which would result in mechanical trouble in the operation of the holder. The forces that may be imposed on the top of the wall have already been discussed.

Some types of gasholder-tank walls are shown in *Fig.* 25.26. The first of these is suitable only in good ground from which there is no uplift due to water pressure. The width of the portion of the footing inside the line of the wall depends on the length of the rest-blocks. A space should be left between the inner edge of the rest-block and the bottom of the sloping slab to allow drainage during erection. The inclined slab is usually reinforced with small-diameter bars. It is often trouble-

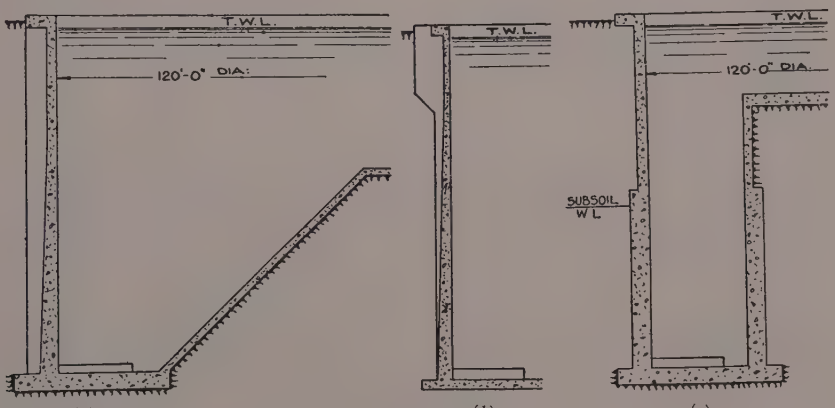


Fig. 25.26.

some to construct, a top form being required, and furnishes a case where it is often cheaper to make a reinforced slab 8 in. thick than to lay a 6-in. reinforced concrete slab on 2 in. or 3 in. of plain concrete.

The design shown in *Fig.* 25.26*b* is a very common type, and except for the inclusion of pilasters and rest-blocks does not differ from a large water or slurry tank.

The design in *Fig.* 25.26*c* is suitable for ground with a low-lying water-table. This consists of a double cylinder, the lower parts of the walls being designed for combined earth and water pressure from without, in addition to water pressure from within. The upper portion is designed for water pressure from within, and sometimes a reduction is made for the whole or a proportion of the active earth

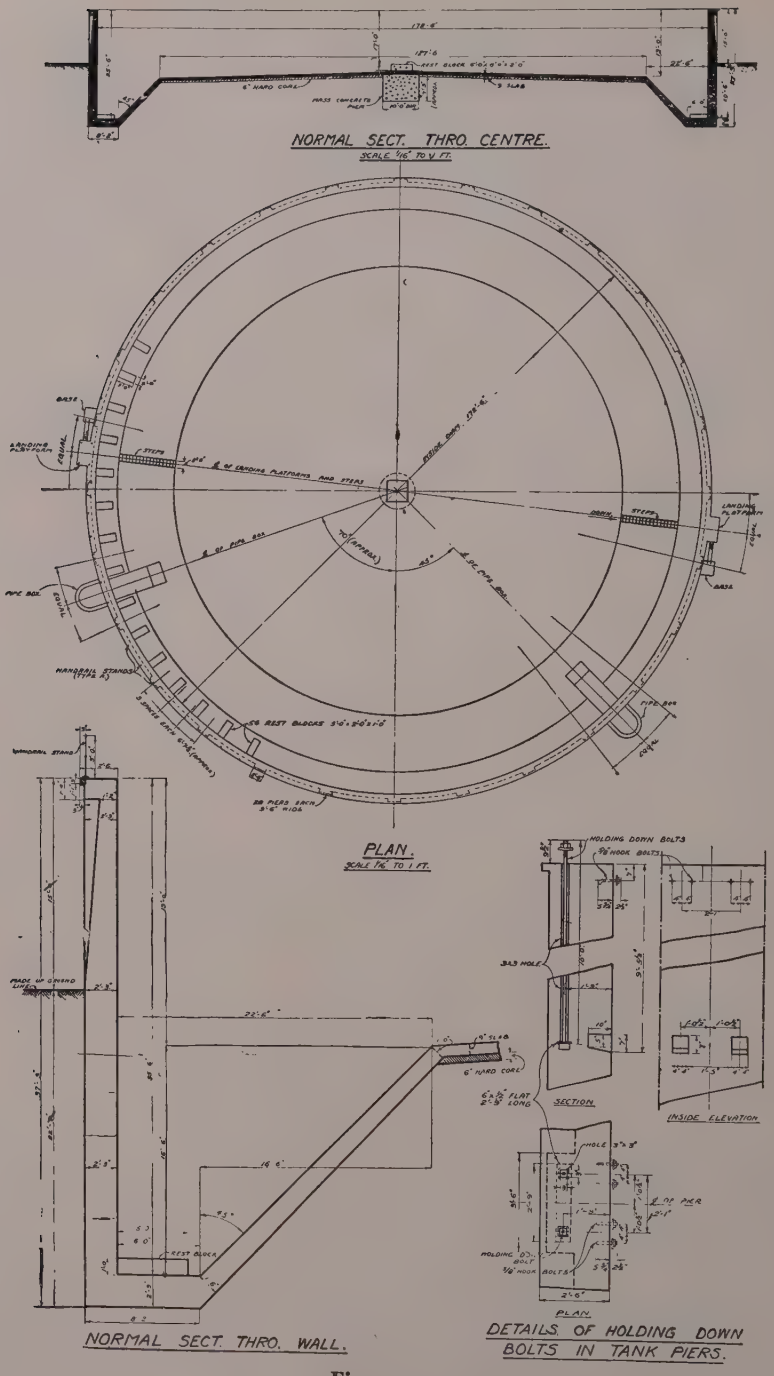

NORMAL SECT. THRO. CENTRE.
SCALE 1/16 TO 1 FT.
PLAN.
SCALE 1/16 TO 1 FT.
NORMAL SECT. THRO. WALL.
DETAILS OF HOLDING DOWN BOLTS IN TANK PIERS.
SECTION.
INSIDE ELEVATION
PLAN.

Fig. 25.27.

pressure from without, but this reduction should not be made for ground such as clay which may shrink away from the wall in dry weather. The upper floor is built above the level of the ground water and is often a 6-in. slab reinforced with $\frac{1}{2}$-in. diam. bars at 12-in. centres at right-angles.

The loads on the rest-blocks are transmitted to the ground by a footing ring, which acts as an inverted beam loaded uniformly, if the settlement is uniform, on the ground and spanning a distance l between the rest-blocks. This is usually designed for a moment $M = \pm\frac{1}{16}Wl^2$, and is reinforced with continuous ring bars in the top and bottom and $\frac{1}{2}$-in. or $\frac{3}{8}$-in. diam. links at about 12-in. centres. The ends of the links should project into the circular slab for a length of about 2 ft. The area of steel in the ring beam may be calculated from the formula $A_{st} = \dfrac{M \times 12}{l_a p_{st}}$, where M is the moment in lb.-ft. The same area (A_{st}) is then used in both top and bottom layers of steel.

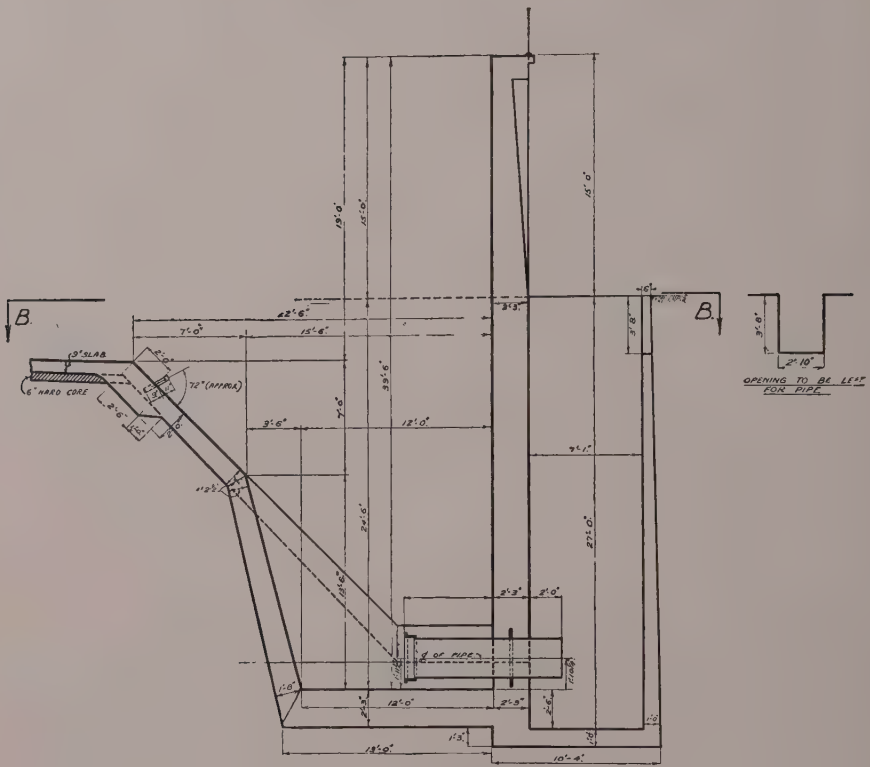
Particular care is required in the design of the pipe pit or well, so that the portion outside the tank is adequately connected to the remainder of the floor. The outer portion of the pipe pit and the main tank are subject to loadings of very different magnitude, and there is a probability of uneven settlement between the two parts of the structure.

Examples

A gasholder tank at Dundee is shown in *Figs.* 25.27 and 25.28. The internal diameter is 172 ft. 6 in. and the depth at the wall 35 ft. 6 in. The wall and the central column, required for erection purposes, were carried down to a solid rock foundation, and the floor of the tank consists of a 9-in. slab on 6 in. of hardcore. For a height of 22 ft. the wall is 27 in. thick; above this level the thickness decreases to 14 in. and there are twenty-eight external pilasters, each 3 ft. 6 in. wide, extending up to the coping which is 16 in. deep. The wall footing is 8 ft. 2 in. wide and 2 ft. 9 in. deep, and carries fifty-six rest-blocks, each 5 ft. by 2 ft. by 1 ft. Connection between the footing and the 9-in. floor slab is by means of a sloping slab, varying from 18 in. thick at the bottom to 12 in. at the top and inclined at 45 deg. to the horizontal. The angle between the centre lines of the pipe pits is 115 deg. This tank was designed to resist the internal pressure partly by hoop steel and partly by cantilever action, and was reinforced by mild steel bars ranging in diameter between $\frac{1}{4}$ in. and 1 in. The concrete mix was 1 : 2 : 4, and the working stresses were limited to 16,000 lb. per sq. in. tension and 600 lb. per sq. in. compression. Steel forms were used for the walls and the concrete was placed by chuting. The walls were coated on the inner side with cement slurry brushed on, and the exposed surfaces were rubbed down and given a coat of cement wash.

Tank with Conical Bottom.—A section through a gasholder tank of 84 ft. diameter with a sloping floor is shown in *Fig.* 25.29 and details of the wall and footing reinforcement are given in *Fig.* 25.30.

Large Gasholder Tanks.—A tank at Sheffield for the East Midlands Gas Board is 230 ft. in diameter and 44 ft. 9 in. deep, with the top 15 ft. 6 in. above ground level, and accommodates a five-lift spirally-guided gasholder of 8,000,000 cu. ft. capacity. For all the reinforced concrete a 1 : 2 : 4 mix was used; the main reinforcement is medium high-tensile steel the design stress in which is 20,000 lb. per sq. in. Continuous rings of reinforcement were formed by welding the ends



DETAILS OF PIPE BOX.

Fig. 25.28.

of the medium high-tensile bars. This resulted in a considerable saving in the weight of steel required.

The main wall, which varies in thickness from 3 ft. at the bottom to 1 ft. at the top, is designed to resist hoop tension only, the restraining moment at the bottom being eliminated by the use of a watertight sliding joint. The wall is covered with a water-resisting membrane held in place by a lightly-reinforced concrete wall 6 in. in thickness. The top of the wall carries the guide-carriages for the gasholder and forms a cantilever walkway 5 ft. wide which also acts as a stiffener resisting horizontal wind loads. The central portion of the tank is an earth 'dumpling,' made watertight with clay puddle after the erection of the gasholder. The top of the dumpling follows the original slope of the ground (about 1 in 12), and the sides are retained by a vertical wall varying in height from 4 ft. to 15 ft. The outer and inner walls are 11 ft. apart and are supported by a slab which also supports the steel 'lifts' of the gasholder when the holder is deflated. The slab is cast on solid rock for part of its length, and the remainder is supported by plain concrete piers and short precast concrete piles driven to rock. Gas inlet and outlet pipes

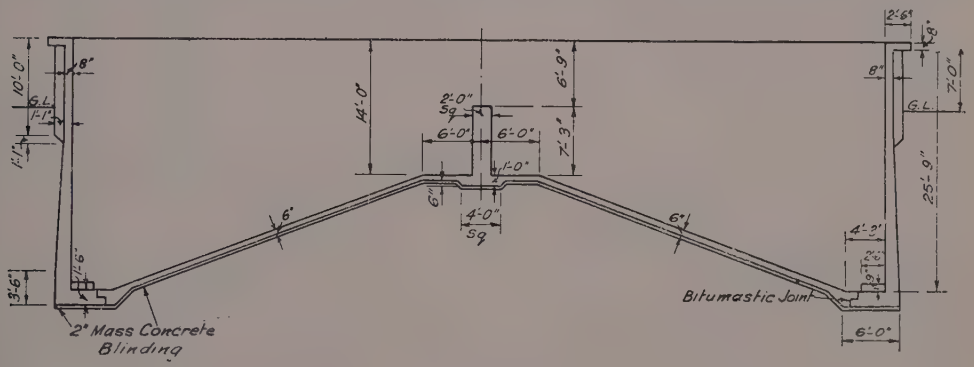


Fig. 25.29.

enter the tank through the bottom, and are carried up outside the tank in reinforced concrete shafts which are separated from the tank wall.

The ground was levelled around the tank, the backfill of soil on the low side being retained by a wall about 20 ft. distant from the tank. This wall is designed as a curved band resisting tension only, with anchorages at each end, and is supported on short columns and footings.

A tank 183 ft. in diameter by 34 ft. deep to accommodate a spirally-guided gasholder with a capacity of 3 million cu. ft. has been built at Cheltenham for the South Western Gas Board. The tank is sunk in the ground to a depth of 24 ft. and has a flat reinforced concrete floor with a single central column to carry the load from the crown of the holder. The floor is 6 in. in thickness, except under the central column and where it forms an annular ring-beam 11 ft. wide which carries the outer wall and the steel 'lifts' when the holder is empty. The whole of the floor is of 1 : 1½ : 3 concrete and, after the holder was constructed and just before the tank was filled with water, the construction joints were covered with aluminium foil and bituminous sheeting to ensure watertightness.

The wall of the tank, of vibrated concrete, varies in thickness from 1 ft. at the top to 2 ft. at the bottom and is designed to resist internal water pressure by circumferential reinforcement which is welded at the laps. The walls are lined

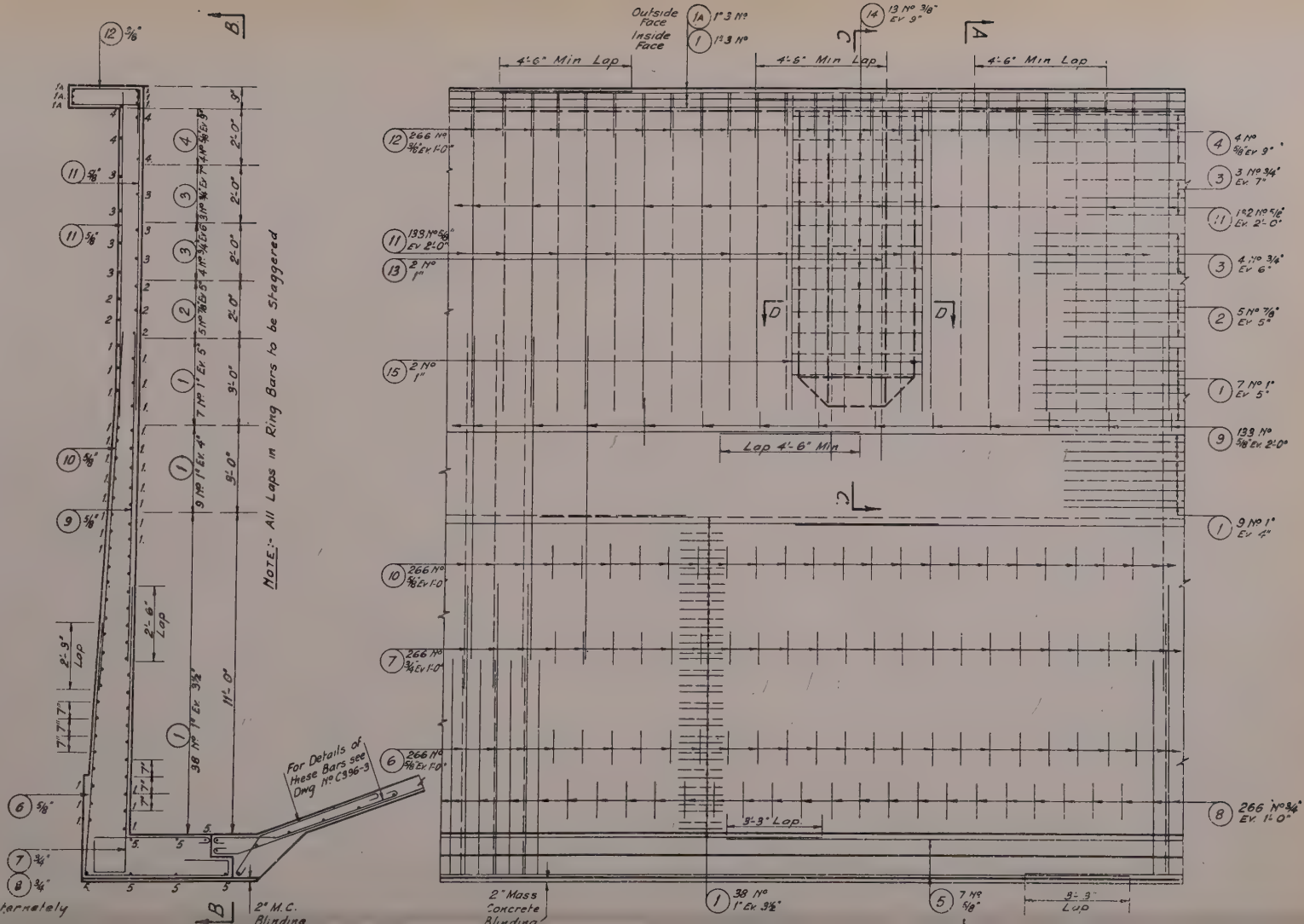

Outside face
Inside face
4'-6" Min Lap
Lap 4'-6" Min
NOTE:- All Laps in Ring Bars to be Staggered
For Details of these Bars see Dwg No C396-3
2'-9" Lap
2'-6" Lap
3'-3" Lap
2" Mass Concrete Blinding
Alternately

Fig. 25.30.

with a waterproof membrane of reinforced bitumen which is protected by a lightly-reinforced concrete wall 6 in. thick. As it was considered that the continuous bitumen membrane would ensure that the tank was watertight, a stress of 18,000 lb.

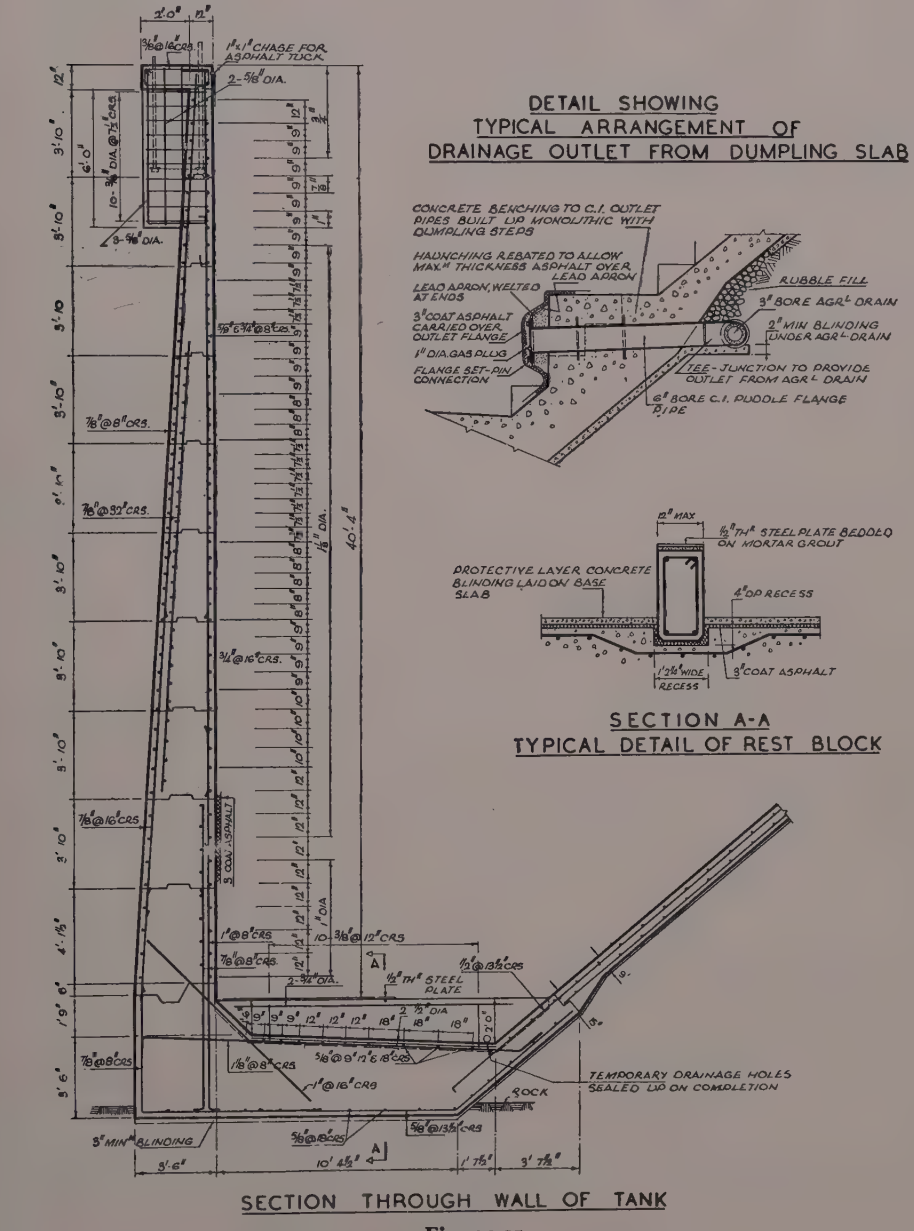

DETAIL SHOWING
TYPICAL ARRANGEMENT OF
DRAINAGE OUTLET FROM DUMPLING SLAB
CONCRETE BENCHING TO C.I. OUTLET PIPES BUILT UP MONOLITHIC WITH DUMPLING STEPS
HAUNCHING REBATED TO ALLOW MAX.M THICKNESS ASPHALT OVER LEAD APRON
LEAD APRON, WELTED AT ENDS
3 COAT ASPHALT CARRIED OVER OUTLET FLANGE
1" DIA. GAS PLUG
FLANGE SET-PIN CONNECTION
RUBBLE FILL
3" BORE AGR.L DRAIN
3" MIN BLINDING UNDER AGR.L DRAIN
TEE - JUNCTION TO PROVIDE OUTLET FROM AGR.L DRAIN
6" BORE C.I. PUDDLE FLANGE PIPE
SECTION A-A
TYPICAL DETAIL OF REST BLOCK
PROTECTIVE LAYER CONCRETE BLINDING LAID ON BASE SLAB
½" TH.K STEEL PLATE BEDDED ON MORTAR GROUT
4" DP RECESS
3 COAT ASPHALT
RECESS
SECTION THROUGH WALL OF TANK
TEMPORARY DRAINAGE HOLES SEALED UP ON COMPLETION
ROCK
BLINDING

Fig. 25.31.

per sq. in. was adopted for the steel reinforcement instead of the more usual 12,000 lb. per sq. in. It was also considered safe to make an allowance for the support provided by the active pressure of the earth. As it was not considered

possible to determine with accuracy the restraint at the bottom of the wall in a tank of this size, the wall is separated from the base of the tank by a sliding joint which incorporates thin steel plates to seal the joint. The top of the wall has a kerb 3 ft. wide which carries the 42 holder carriages, the bolts for which are enclosed in pilasters spaced equally around the tank. The kerb resists the horizontal component of wind loads transmitted to the carriages by the steel superstructure.

Details of a gasholder tank of 232 ft. internal diameter at Bristol are given in *Fig.* 25.31.

CHAPTER TWENTY-SIX

NOTES ON CONSTRUCTION

The basic principles of construction that apply to all reinforced concrete work will apply also to water-retaining structures. Only points which apply with special emphasis to watertight work are discussed here.

Storage of Industrial Liquids

Many substances are not injurious to concrete, for example, those known as bases and the compounds of carbon (the paraffins). Others are distinctly harmful, and some may be, but are not always, destructive. Nearly all acids and many salts, especially sulphates, can produce harmful effects. Examples of these are (*a*) sulphuric, hydrochloric, nitric, hydrofluoric, carbonic, oleic, saccharic, and acetic acids, and phenol, and (*b*) magnesium sulphate (in sea-water), magnesium chloride, sodium sulphate, linseed oil, olive oil, butter, cocoa butter, fruit juices, milk, and all other fatty oils. Sulphuric acid in accumulator rooms and acetic acid at artificial silk works are frequently responsible for the need of protective coatings on structural materials.

Substances which are harmless to concrete include sodium chloride, calcium chloride, oxalic acid (the only harmless acid), sodium hydrate, sal ammoniac, lime water, mineral oils, benzol, paraffin, Vaseline, etc.

Protective measures include the provision of linings of glass plates jointed with acid-resisting cement or wax, coatings of acid-resisting asphalt or paraffin wax, lead linings, and mixtures of pitch and tar. In some cases, for example at breweries, it is important not to use a lining which will give the liquid a peculiar flavour, alter its colour, or introduce any smell of chemicals. Wax enamelling on the rough (unrendered) concrete surfaces has been found to be satisfactory and does not interfere with the stability of the beer. If the tank has been rendered, the surface must not be heated too much when putting on the enamel with the blow-pipe, otherwise the rendering will flake off.

So many contradictory statements have been made concerning the storage of fuel oil in reinforced concrete tanks that an engineer may be bewildered when he is faced with the problem. The views expressed in the following paragraph are those of an engineer with great experience in this work.

It is the policy of oil companies in the Far East not to use concrete tanks generally for storing heavy oil. The reason for this is not that concrete is damaged by the oil or the oil by the concrete, but that steel tanks are easily dismantled and removed elsewhere when necessary, whereas concrete tanks are essentially fixtures. These companies have found that fuel oil does not harm the concrete in those tanks they have built or in concrete containers in which tins of oil are stored and leak because of bad soldering. On the contrary, cement wash is used to protect the bottoms of the steel tanks from rust caused by water separating from the oil. Cement does not harm the oil, and is used as a $\frac{1}{2}$-in. spun lining in old pitted metal pipes in the pumping system. The polished surface of the cement is found to give a low frictional resistance to pumping.

Some design details of a 40-ft. diameter fuel oil tank are given in *Fig.* 26.1. The concrete was mixed 1 : 1½ : 3 by volume for the floor, walls, columns, and footings; 1 : 2 : 4 by volume for the roof slab and beams; and 1 : 3 : 6 for plain concrete. A 4-in. by 16-gauge metal strip, well lapped at its ends, was cast in at all horizontal construction joints.

Even when very dense concrete is used for tanks to contain petrol, losses have occurred due to the volatile liquid percolating through the concrete. In a system

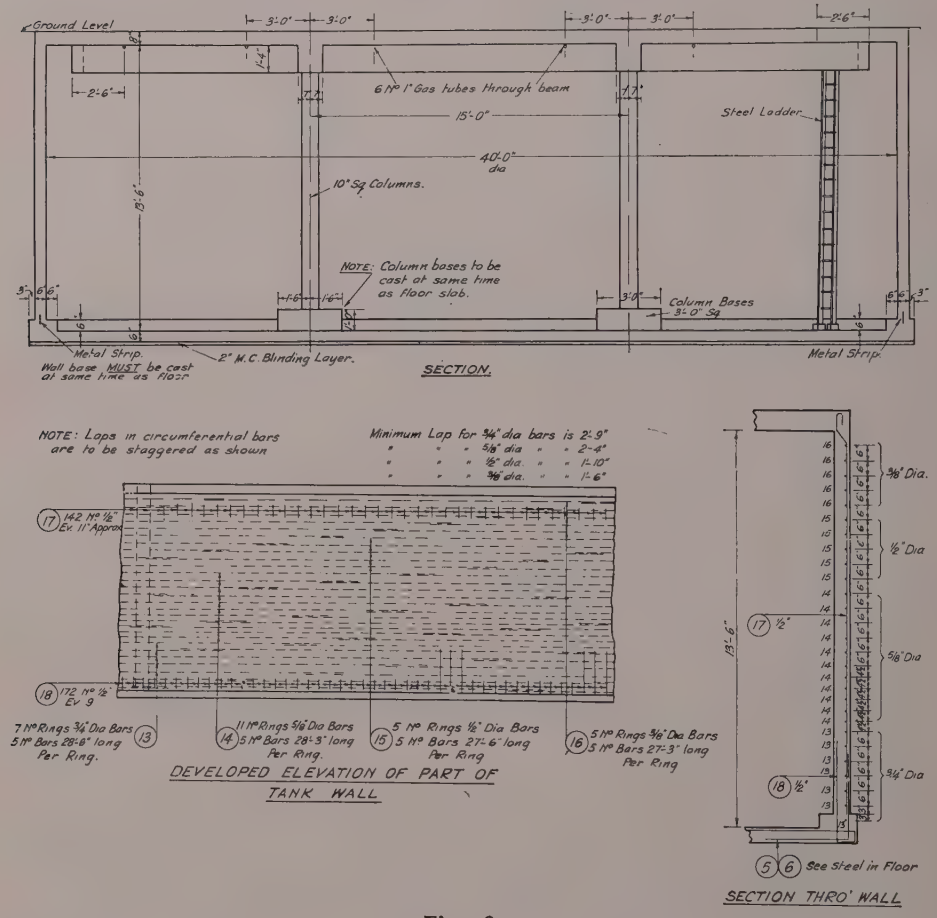

Ground Level
6 Nº 1" Gas tubes through beam
Steel Ladder
15'-0"
40'-0" dia
10" Sq Columns.
NOTE: Column bases to be cast at same time as Floor slab.
Column Bases 3'-0" Sq
Metal Strip.
2" W.C. Blinding Layer.
Wall base MUST be cast at same time as Floor
SECTION.
NOTE: Laps in circumferential bars are to be staggered as shown
Minimum Lap for ¾" dia bars is 2'-9"
5/8" dia " 2'-4"
½" dia. " 1'-10"
⅜" dia. " 1'-6"
17 142 Nº ½" Ex. 11" Approx
18 172 Nº ½" Ex 9
7 Nº Rings ¾" Dia Bars 5 Nº Bars 28'-8" long Per Ring.
11 Nº Rings 5/8" Dia Bars 5 Nº Bars 28'-3" long Per Ring.
5 Nº Rings ½" Dia Bars 5 Nº Bars 27'-6" long Per Ring
5 Nº Rings 3/8" Dia Bars 5 Nº Bars 27'-3" long Per Ring
DEVELOPED ELEVATION OF PART OF TANK WALL
13'-6"
3/8" Dia.
½" Dia
5/8" Dia
¾" Dia
See Steel in Floor
SECTION THRO' WALL

Fig. 26.1.

known as ' La Paroi Hydraulique ', a double wall is used, the space of about 1½ in. between the leaves being filled with hollow blocks made of porous concrete. The cavities extend vertically in the walls. A false bottom is also provided. The cavities in the walls and floor are kept filled with water, and the hydrostatic pressure of the water prevents leakage of the contents of the tank.

In a system devised by M. E. Miozzi, of Venice, the petrol tank is surrounded by a larger and deeper tank which is filled with water and so prevents seepage of petrol through the sides and top. The bottom of the petrol tank is filled with water, on which the petrol floats.

After experimental work in advance of the construction of more than 2,000 concrete oil-storage tanks of capacities ranging from 25,000 to 2,500,000 gallons, the United States Navy Department in the year 1943 adopted the following linings and protective measures. For heavy fuel oil, four coats of silicate of soda on the inside. For diesel oils and petrol, three different types of lining have been developed, namely a vinylite plastic painted on the interior, sheet thiokol latex (an aqueous suspension of polysulphide of rubber) cemented to the interior, and thiokol in latex form and a layer of 4-oz. cotton fabric; the latex thiokol is painted on the inside of the tank, the fabric is embedded in the latex, and a coat of thiokol is applied over the fabric. The tanks were prestressed. During three years of use, no leakage was noticeable and when the tanks were drained the following results were reported: Exposed metal was slightly corroded. Exposed concrete in the roof showed no sign of cracking or spalling. The lining was slightly discoloured but otherwise it was intact. The fabric was tight, and there was no indication of loosening at the joints.

It has been proposed that reinforced concrete oil tanks be lined with a thin continuous sheet of high-purity aluminium, welded at the joints and laps.

Storage of Hot Liquids

It should be remembered that circular tanks ' breathe ' more easily than rectangular ones and thin walls ' breathe ' more easily than thick ones. The radial thickness of the wall of a circular tank is constant. However, if the four walls of a rectangular tank are 12 in. in thickness, they are 12 in. thick radially at the centres of the sides but are $\sqrt{2} \times 12$ or 17 in. in thickness, at the corners. A warm circular tank exposed to a cold wind causes less discontinuity in the air stream lines, loses less heat and loses it more uniformly. A cold wind eddying round a warm square tank chills the corners to a temperature many degrees below the temperature in the centre of the sides. This is at once apparent if the hand is pressed against different parts of the wall. The difference is sufficient to open horizontal cracks across the corners.

If the floor is anchored to the foundation or if it is colder than the walls, a severe ' restraint ' effect may be set up when the walls warm up and expand. The extension caused by a rise in temperature of 11 deg. F. (if $E_c = 3{,}000{,}000$ lb. per sq. in.) is the same as the extension caused by a tensile stress of 200 lb. per sq. in. The floor of the tank should be designed to slide freely over the foundation so that it can expand and contract in unison with the walls.

Tanks to hold warm liquids should at least be protected from the direct impact of cold winds even if they cannot be insulated effectively or housed in warm buildings.

Storage of Very Cold Liquids

The necessity for the inground storage of liquid natural gas at a temperature of −260 deg. F. (−160 deg. C.) has raised the problem of the strength of reinforced concrete at these low temperatures. Although much further investigation is necessary it seems that the concrete maintains its strength at such temperatures. Mild steel also retains its strength but tends to become brittle. High-nickel steel remains ductile but it is difficult to obtain bars of a length suitable for large tanks.

Preliminary experiments indicate that the adhesion between steel and concrete may increase at very low temperatures.

Concrete Materials and Placing

Materials.—The use of rapid-hardening Portland cement is seldom necessary in the construction of reservoirs. Also, it is not advisable to use it because of the extra shrinkage of concrete made with rapid-hardening Portland cement due to the additional heat generated during the setting and hardening period, and to the extra mixing water necessary to produce the required workability owing to the finer grinding of this type of cement. Sudden changes of temperature, due for example to the early removal of timber shutters from hot concrete on a cold day, often cause cracks that may result in leaks. The less heat generated in the early stages the better.

Rounded aggregate makes a more workable and therefore a more watertight concrete than crushed aggregate. Of two tanks of identical design built with the same foremen on the same site the tank built with concrete containing natural rounded shingle aggregate was more watertight than one built of crushed granite aggregate.

Proportions.—A mixture of 1 cwt. of cement, $1\frac{7}{8}$ cu. ft. of sand, and $3\frac{3}{4}$ cu. ft. of coarse aggregate, that is a nominal $1 : 1\frac{1}{2} : 3$ mix, is commonly used for floors, walls, and roof. In CP.2007:1960 proportions of 1 cwt.: 2 cu. ft.: 4 cu. ft. are suggested, that is a nominal $1 : 1{\cdot}6 : 3{\cdot}2$ mix. The decrease in permeability obtained by the use of additional Portland cement has already been mentioned. Some engineers add hydrated lime to make concrete more workable and denser. An example is the mix specified for the reinforced concrete filters of the Metropolitan Water Board at Kempton Park, in which the proportions were 200 lb. of Portland cement to $4\frac{1}{4}$ cu. ft. of sand and $8\frac{1}{2}$ cu. ft. of shingle plus hydrated lime in the proportion of 8 lb. to each 1 cwt. of cement. It has been said that the substitution of a $1 : 1\frac{1}{2} : 3$ concrete for a $1 : 2 : 4$ mix is not desirable in reservoir construction because the richer mixture shrinks more; however, this argument is unsound since $1 : 2 : 4$ concrete requires a higher water-cement ratio with consequent extra shrinkage which annuls any smaller shrinkage due to the smaller cement content.

The amount of mixing water depends on the workability required rather than on the strength of test cubes. Not only must it be possible to compact thoroughly each layer or batch of concrete, but it must also be possible to knead each layer or batch into the preceding layer or batch. Otherwise the structure may tend to become a collection of separate units with joints between them, through which water may leak. Whatever means are provided for compacting the concrete, enough water must be added to ensure a solid and monolithic structure. Careful supervision of the first section to be concreted and careful inspection when the shuttering is stripped should show whether too little or too much water has been used, and neither the water-cement ratio nor the slump should be definitely fixed until the first section of the work has been completed. For watertight work a sufficiently workable concrete is essential even if it means using different, smaller, or less coarse aggregate, or increasing the cement content to compensate for the increased mixing water.

Transporting Concrete.—In reservoir construction relatively small amounts of concrete have to be transported over a considerable distance with only a limited amount of lifting. Whatever type of plant is provided, the construction of the floor should be a simple matter. The roof also is simple, since the concrete can be wheeled on runways supported from the shuttering. Each part of a wall should be constructed immediately after the part of the floor beneath it, and this generally

means that the shuttering for the roof (if any) is not in position sufficiently soon to serve as a high-level runway for concreting the walls. The mixing plant may be at floor level and the concrete barrowed to the base of the wall and then lifted by hoist or mobile crane. Alternatively the mixing plant may be at roof level and a high-level runway provided at the level of the top of the wall. A third method is to use plant which is capable of both lifting and transporting, such as a chuting plant or concrete pump. The choice depends on the size of the work, the general levels of the site, and the type of plant that is readily available.

Small contracts have often to be undertaken with plant that is easily available, however unsuitable it may be. On larger works it is usual to provide reasonably new standard plant well suited for the work, while in the case of very large contracts the plant is often specially designed for the work. In *Fig.* 26.2 a monorail transporter, of a type that is now frequently employed for distributing concrete, mounted on a gantry of tubular scaffolding is shown. In *Figs.* 26.3 and 26.4 a chuting plant at some filter tanks constructed for the Metropolitan Water Board is illustrated.

Fig. 26.2.

The aggregates were dumped alongside the tower and raised by a grab to timber hoppers situated above the mixer. A tower or mast is not always the most economical plant to use, particularly where wages are low. A large daily output of concrete is necessary to achieve economy with a tower, which in turn requires a considerable area of shuttering when walls are being concreted. Assuming that the mixer produces 40 cu. yd. daily and that the walls are 12 in. in thickness, 240 sq. yd. of shutters must be erected each day if the plant is to be kept continuously employed. Chuting plants such as that shown in *Figs.* 26.3 and 26.4 were once in common use, but are now seldom seen. They can be successfully used only if both the designer and contractor understand their limitations. Rounded aggregate and a reasonably wet mix are essential. It is also advisable to limit the amount of coarse aggregate so that this does not exceed the combined volumes of the sand and cement. A 1 : $1\frac{1}{2}$: $2\frac{1}{2}$ mix can be chuted safely with normal aggregates, while a 1 : $1\frac{1}{2}$: 3 mix may not be suitable. A concrete pump, on the other hand, cannot be successfully used to transport very wet or very harsh concrete, but is well suited to the rich workable mix required for watertight work.

Fig. 26.3.

Fig. 26.4.

Placing and Compacting Concrete by Hand.—In *Fig.* 26.5 an ideal conception of the process of concreting a wall when the concrete is compacted by hand tamping is shown. The concrete is placed in successive layers A, B, C, etc., each about 6 in. high. The minimum initial setting time for Portland cement is forty-five minutes, and it might be thought sufficient to tamp each layer within forty-five minutes of first wetting it, but for watertight work this is not enough. As already pointed out, each layer must be rodded down into and knitted with the layer below. Thus layer D must be tamped on to and into layer C before layer C sets, that is within forty-five minutes of first wetting layer C. In effect this means that one layer must be placed every twenty minutes (in normal

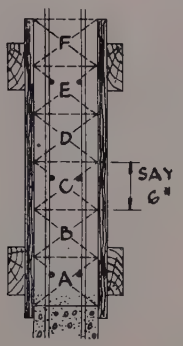


Fig. 26.5.

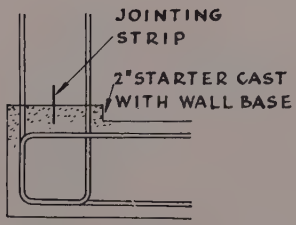


Fig. 26.6.

temperatures). If each layer is 6 in. high this means placing a height of 18 ins. per hour. With a 12-in. wall concreted in sections 50 ft. long this is a minimum of 75 cu. ft. or, say, 3 cu. yd. per hour. The fact that every layer must be placed before the preceding layer sets must be particularly borne in mind if it is proposed to use ready-mixed concrete. There is no practical limit to the height of wall that may be concreted in one operation provided that the men entrusted with the tamping can easily see and easily reach the surface of the concrete throughout the operation. Only in the case of thick walls (say, 2 ft. and upwards) can tipping wagons, concrete carts, pumps, or crane skips be discharged directly into the shutters. In the case of thinner walls the concrete must be dumped on to a banker and distributed (usually by shovels) in layers sufficiently thin to ensure thorough compaction.

Placing and Compacting Concrete Mechanically.—On small contracts with little or no technical supervision mechanical consolidation is best not attempted.

The consistency of the concrete must be carefully and continuously regulated. If external vibrators are employed the shuttering must be adequately strong but at the same time sufficiently flexible to transmit the vibrations to the concrete; it should also be uniformly flexible, otherwise it tends to fall into nodes and antinodes with the result that some of the concrete is over-vibrated and some honeycombed. This may mean altering the positions of the vibrators as the work proceeds. Usually the most difficult part to compact by external vibrators is the lowest part of each lift of wall. Since the horizontal construction joints are, in any case, the weakest sections of the wall, honeycombing at this level is likely to be serious.

When internal vibrators are used, a regular programme of insertion should be prepared. Haphazard dipping of the machines into the centre of a wall may produce a line of ' puddles ' along the middle with extensive honeycoming at the

faces. Compared with hand ramming, the rate of concreting should be hastened so that no concrete more than one hour old is subjected to appreciable vibration. In all cases the designer should insist on inspecting the first section of wall immediately the shuttering is removed and before any making-good is attempted.

Horizontal Construction Joints in Walls.—Most walls are concreted in lifts of about 4 ft., and one of the major problems is to make a watertight joint between the lifts. In all cases the surface of the lower lift should be cleaned. If the concrete is only one or two days old it is dangerous to hack the surface with a pick, and wire brushing and hosing are the best treatment. If the concrete is more than seven days old it may be hacked, and if a month old a pneumatic chisel may be used. It is common to insert a strip of sheet steel, 4 in. to 6 in. wide and of about 20 gauge, in the construction joint at the base of the wall (*Fig.* 26.6), and some designers use these, or purpose-made plastic strips, at all horizontal joints in a wall. Before beginning to concrete the upper lift the surface of the lower lift should be wetted and a layer of 1 : 1 : 2 concrete about 3 in. thick spread over it and thoroughly tamped down. In the I.C.E. Code the use of a layer of 1 : 1½ sand and cement is suggested instead of 1 : 1 : 2 concrete, but the main point is to ensure that this layer is thoroughly worked down on to the surface of the lower lift. If a wall or column splays out at the top, a joint should be made at the bottom of the splay and the lower part allowed to harden and cool before concreting proceeds.

Vertical Construction Joints in Walls.—As a rule there is less risk of leakage through vertical joints than through horizontal joints. The surface should be cleaned and wetted. The new concrete should be spaded away from the old face and then spaded back on to it. It is often possible to provide sufficient shuttering to enable the concrete from one expansion joint to the next to be placed in one operation, and thus avoid vertical construction joints in the walls.

If there are severe stresses across the horizontal construction joints, the insertion of metal or plastic jointing strips might be considered here also. The provision of jointing strips should always be regarded as an additional safeguard and not as an excuse for slack supervision or a low standard of workmanship.

Curing.—The importance of curing the freshly-placed concrete has already been mentioned. If the wall is above ground and exposed to the sun and the wind extra precautions must be taken.

It is customary to assume that the shrinkage of air-dried concrete may be a maximum of 1/2,000th part of its length. An approximate idea of the effect of shrinkage may be arrived at as follows. Assuming that the coefficient of expansion of concrete is 0·000006 per deg. F., a rise of 50 deg. in temperature would cause a 100-ft. length of wall to expand 0·36 in., or nearly ⅜ in. It is not always realized how great is the contraction of air-hardened concrete, and adequate precautions are not always taken to cure the concrete. If, for example, concrete is hardened in air, a length of 100 ft. contracts about $\frac{100 \times 12}{2{,}000} = 0{\cdot}6$ in. (assuming a high value of the coefficient of shrinkage). This shrinkage is equivalent in magnitude to the contraction which would occur owing to a fall in temperature of $50 \times \frac{0{\cdot}60}{0{\cdot}36} = 83$ deg. F. Although no engineer would willingly subject new concrete to a temperature range of 80 deg., the material is sometimes so improperly cured that the effects of shrinkage are of the same order.

Tests have shown that permeability is very much less at twenty-eight days for

concrete cured in a moist condition for seven days, than for similar concrete moist-cured for only three days. Specimens cured in a moist condition for the whole period of twenty-eight days were far less permeable than those cured for the shorter periods. Lack of watertightness is often due to the use of excessively wet or very stiff concrete; when stiff mixes are used, plasticisers help to improve workability and thus offset the defects due to bad workmanship.

Very wet concrete must not be used for concrete that is to be bonded with older work, as this is often a cause of separation and planes of leakage. If new concrete is to be joined to existing work, a pneumatic pick is useful for preparing the surface.

If rapid-hardening Portland cement is used it should be remembered that the same total quantity of heat is developed in rapid-hardening and in ordinary Portland cement while setting and hardening, and therefore more water must be applied during the initial stages of hardening of rapid-hardening Portland cement concrete if the curing is to be adequate.

There are now on the market curing agents which can be painted on the surface of newly-set concrete and which retain the moisture by preventing evaporation. In a hot dry climate where water is scarce these agents are very useful.

Formwork

Full details of the design and construction of formwork will be found in ' Formwork ' by Wynn and Manning. Only the main basic points are touched on here.

Wall Forms.—In addition to the usual requirements, the wall formwork for watertight work must be such as not to interfere with the placing of the concrete. The depth of the forms must be restricted (unless the wall is sufficiently thick for a man to work inside) so that the surface of the concrete can be clearly seen and easily reached. The number of shuttering ties passing through the wall must be restricted, and they must be of a type that will not cause a path of leakage through the wall. Restriction of the number of ties may result in an increase in the cost, and if it is decided to limit or forbid the use of ties this should be made clear in the specification and, if necessary, repeated in the bill of quantities.

Nearly all formwork consists of facework made up in panels, supported by or kept in alignment by some kind of framing such as vertical soldiers, walings, raking shores, struts, or clamps. Unless mechanical means are provided for moving them the panels must be restricted in size and weight, particularly if they have to be erected by hand on scaffolding well above floor level.

Standard steel wall forms 2 ft. by 2 ft. in size are available; these are keyed or clamped together and provided with steel aligners. They are usually supported by numerous wire ties, usually two to four strands of 12-gauge or 14-gauge soft steel wire, which pass through and are left in the wall; the concrete at their ends is scraped away, and the protruding ends are cut off, turned back, and driven into the concrete, and the surface made good. This work may be badly done or overlooked, with very unsatisfactory results. In any case closely-spaced wire ties interfere with ramming and tamping and with the use of internal vibrators, and for this reason some engineers forbid the use of forms of this type for watertight work.

Timber forms, particularly if vibrators are used, should be made of tongued-and-grooved boards screwed to cleats. If timber forms are fixed with steel bolts of $\frac{5}{8}$-in. diameter, the bolts must be straight, greased before use, and given a turn some twelve or eighteen hours after the concrete is placed. When the shuttering

is struck the bolts are driven out, leaving a clean round hole. A simple way to plug these holes is to drive in a sound soft cork from each side and fill the rest of the hole with sand and cement mortar.

Although they are preferable to wire ties, many engineers object to bolts passing through a wall. As an alternative, bolts made in three (or five) pieces can be used, so that when the shuttering is struck the two (or four) outer pieces can be unscrewed and removed and the central piece left permanently embedded. The central piece is of such a length that its ends are recessed about 1 in. from the face of the concrete. With bolts of this type care must be taken to pack the concrete tightly around them and the holes must be carefully filled with mortar or synthetic resin; the fillings may be painted with tar or bitumen.

If all internal ties are forbidden then external struts may be used. These are

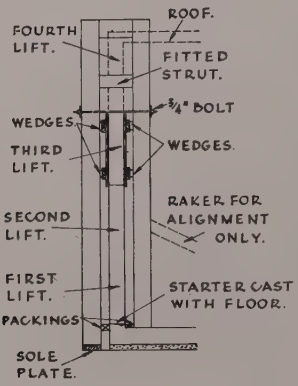


Fig. 26.7.

simple if the wall is built in a timbered or sheeted trench. If the wall stands free above floor level then raking struts are necessary with their lower ends resting in temporary pockets left in the floor or supported on timber sleepers embedded in the ground. If a raking strut is steeper than 1 (vertical) in 3 (horizontal) its upper end must be tied down or weighted or it may overturn; for walls above 10 ft. high external struts soon become very long and cumbersome.

The 'hairpin' method is shown in principle in *Fig.* 26.7. A bolt is passed through the soldiers immediately above each lift of wall and a packing-piece inserted between them about a foot higher up. The soldiers must be sufficiently stiff to prevent the bottom of the shutters being forced out due to the pressure of the wet concrete, and the arrangement shown would be economical only where timber is cheap. Alternatively a steel clamp would be used with tubular steel scaffolding to carry the weight of the forms.

Wall Formwork in Trench.—When a wall is constructed below ground level in a timbered (or sheeted) trench the struts supporting the timber always present a major obstacle to fixing the steel, erecting the formwork, and possibly to concreting. In the case of a retaining wall that need not be watertight some of the struts may pass through the forms and be left in place until the wall is sufficiently strong to resist the lateral pressure. If these struts are made of precast concrete, only the visible ends need to be cut off: but in watertight work the struts should be removed before concreting each lift. This may entail much re-strutting and transference of lateral pressure on to the wall before it is strong enough to resist it.

It may therefore be cheaper and sounder to use steel trench-sheeting with only a single line of struts instead of simple timbering with two or three lines of struts, as the higher initial cost of the sheeting is offset by a reduction of labour costs on the reinforcement, formwork, and concreting. In a deep trench in wet ground steel sheet-piling may often be avoided if the contractor has skilled timbermen. But any initial saving is almost certain to be heavily outweighed by subsequent increases in labour and overhead costs, as well-driven steel sheet-piling is of great assistance for work below ground-water level.

Formwork on Very Large Contracts.—The formwork on very small contracts is often made of timber that has already been used. For most work, however, new timber or steel forms are available. On very large works where the same formwork can be re-used many times, purpose-made steel forms especially

Fig. 26.8.

designed for the work can be economically employed, the cost being more than recovered by the saving in labour; arrangements should also be made for mechanical transport or lifting of the forms and for levelling, aligning, and stripping them. An example is shown in *Fig.* 26.8. With the present tendency towards mechanization there is a trend towards more frequent use of such purpose-made forms.

Continuously-moving Forms.—It is now standard practice to construct the walls of silos with forms that are continuously jacked up on 1-in. diameter steel bars set in the walls. This method has not been widely adopted for tanks and reservoirs, but the absence of horizontal construction joints is a point in its favour, and the fact that the concrete is never more than 6 in. below the top of the form

is another. The method has been used for constructing the walls of slurry tanks. Continuously-moving forms are readily applicable only to walls of constant thickness.

Underground Tanks as Caissons.—Circular tanks up to 50 ft. in diameter, square tanks up to 50 ft. square, and tanks of similar plan area of simple and symmetrical shape may be sunk as open-topped caissons. The walls should be about 2 ft. in thickness to provide sufficient weight to overcome earth friction, which is about 2 cwt. per square foot for all types of soil, the stiffer soils being more easily and widely undercut. The shuttering for the walls may be hung from frames above ground level, swung into position, and filled with concrete. At this stage excavation should be stopped until the concrete has set and the shutters struck and swung clear. Excavation is then recommenced and the caisson sunk through a depth equal to a lift of shutters. The shuttering is then swung back and the process repeated.

The final sealing of the floor slab in waterlogged ground can usually be done by building in a length of 30-in. diameter cast-iron flanged pipe to act as a sump, the pipe being finally closed by a blank flange-plate.

The successful design, construction and sinking of a caisson depends more on understanding and coping with local ground conditions than on the calculation of

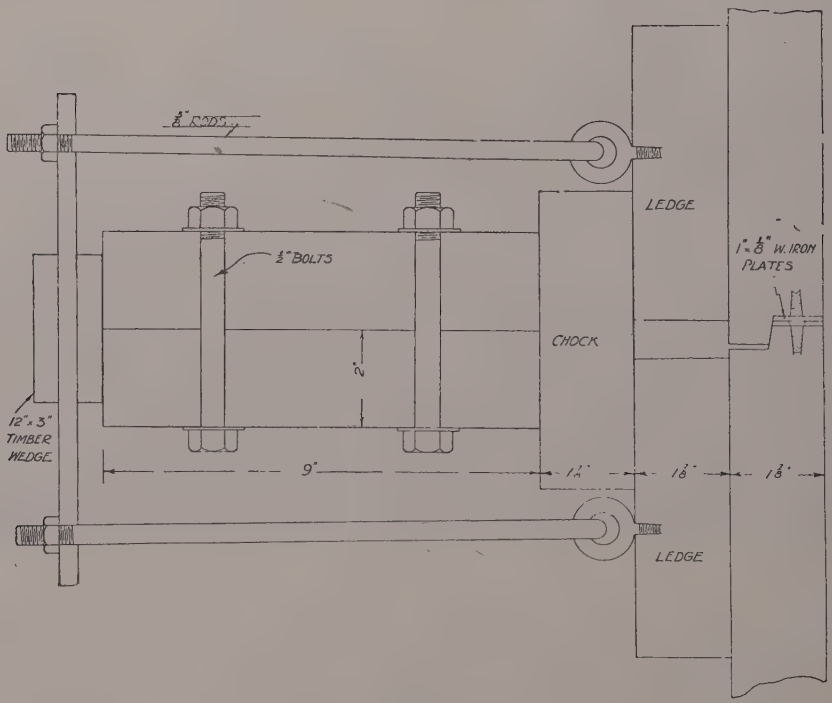


Fig. 26.9.

moments, thrusts and shearing forces. The subject is therefore more a matter of foundation design than the design of a water-retaining structure. For further information and examples the reader is referred to the author's 'Foundations'.

Formwork for Roof.—The spacing of columns and the beams (if any) in a reservoir roof is dictated only by questions of economy, and if the contractor has

formwork available from other contracts it may be economical to introduce small modifications in the design so that the old forms can be used with little alteration. If 2-ft. by 2-ft. standard steel forms are available it is most convenient if all the panels for the soffit of the roof are made an exact multiple of 2 ft. (measured between beam boxes).

Examples of Wall Forms.—In the construction of plain concrete storage reservoirs at Aberdeen the walls were built in alternate sections 30 ft. long. Each section was completed in one day. Travelling steel forms were employed, and the

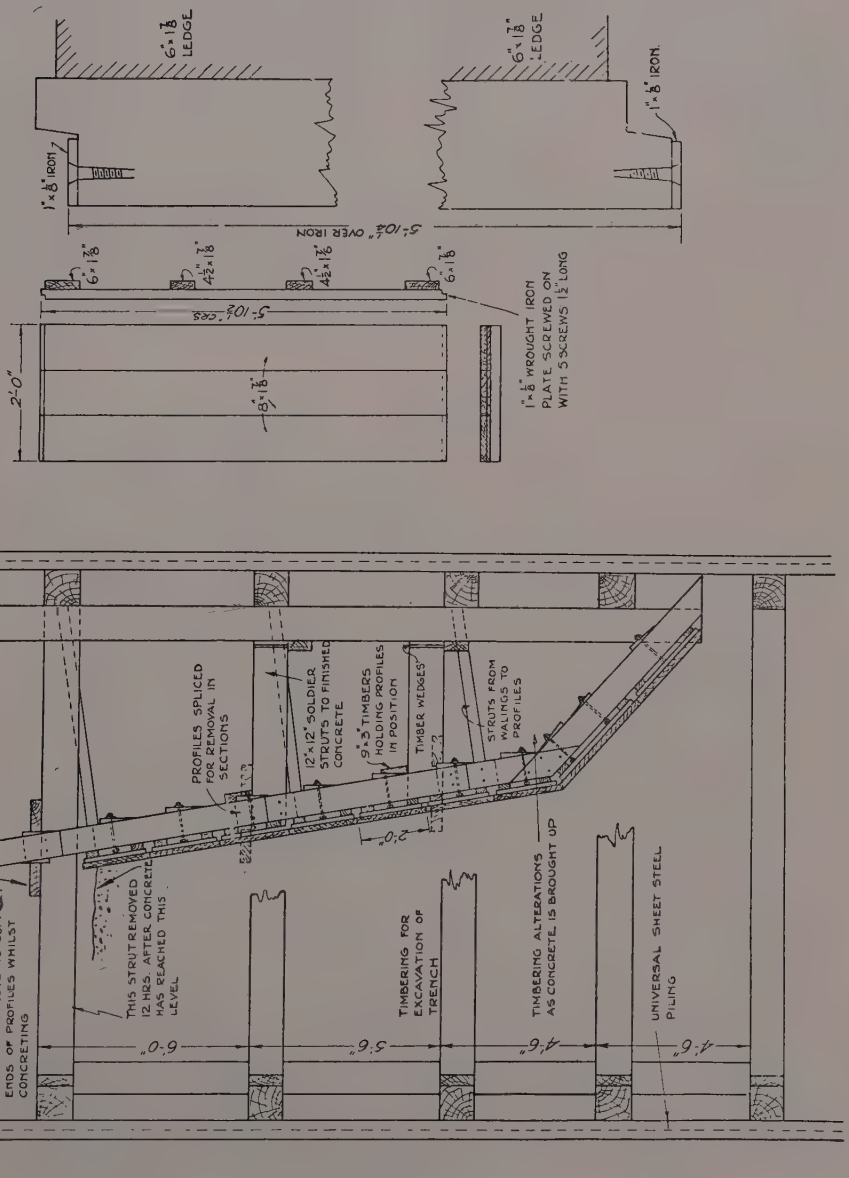

6"×1⅞" LEDGE
1"×8" IRON
5'-10½" OVER IRON
6"×1⅞"
4½"×1⅞"
1"×8" WROUGHT IRON PLATE SCREWED ON WITH 5 SCREWS 12" LONG
2'-0"
8"×1⅞"
PROFILES SPLICED FOR REMOVAL IN SECTIONS
12"×12" SOLDIER STRUTS TO FINISHED CONCRETE
9"×3" TIMBERS HOLDING PROFILES IN POSITION
TIMBER WEDGES
STRUTS FROM PILING TO PROFILES
TEMPORARY 11"×4" TIMBERS SPIKED TO STRUTS TO SUPPORT ENDS OF PROFILES WHILST CONCRETING
THIS STRUT REMOVED 12 HRS. AFTER CONCRETE HAS REACHED THIS LEVEL
TIMBERING FOR EXCAVATION OF TRENCH
TIMBERING ALTERATIONS AS CONCRETE IS BROUGHT UP
UNIVERSAL SHEET STEEL PILING
0'-9"
9'-5"
9'-6"
9'-4"

Fig. 26.10.

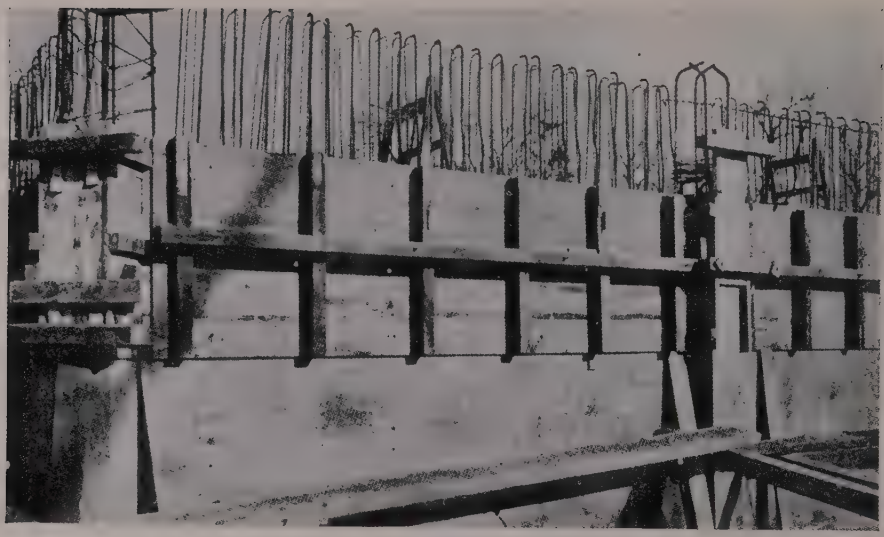


Fig. 26.11.

Fig. 26.12.

concrete was deposited by a combination of hoist, hopper, and chute (*Fig.* 26.8). Along with the concrete a 1½-in. facing of cement mortar was worked on the water face of the walls. The wall sections have U-shaped recesses at the ends and vertical steel plates across the joints. The ends of the sections were painted with a thick coat of mixed tar and pitch before filling in the intermediate sections. The floor was covered with asphalt and this was worked into a groove near the toe of the walls.

At the Arkley Reservoir, Barnet, the 10-in. square columns were precast and lifted into recesses left in the footing blocks which projected above the floor level. Wedges were used to hold the columns in position and were knocked out after grouting the connection. This method of construction is advantageous in reducing the cost of formwork and scaffolding and allows the bearers for the roof forms to be bolted to the concrete columns.

As a general rule the type of formwork suitable for plain concrete retaining walls is also suitable when constructing a plain concrete reservoir. Two examples show improved methods of constructing formwork which contain points of interest to waterworks' contractors.

For the construction of a plain concrete wall at the Grosvenor Canal reconstruction the contractors used vertical steel channels instead of timber soldiers. These steel channels were drilled through both flanges at 6-in. centres, and steel plates, with small steel channels riveted to their upper and lower edges and stiffened by vertical steel angles, one at each edge and one at the middle of the plate's length, constituted the panels. The outer steel angles were drilled to match the holes in the flanges of the vertical channels, and all the parts were interchangeable. The channels were either strutted off the existing work or tied back into the concrete by ⅜-in. diam. threaded rods passing through holes in the web of the vertical channel and a packing piece in front. When the forms were stripped, the ⅜-in. diam. rods were cut off at the surface of the wall.

An ingenious arrangement of wall forms used on the construction of a plain concrete wall is shown in *Figs.* 26.9 and 26.10. The panels are 2 ft. wide, built up from 8-in. by 1⅞-in. boards and stiffened with 6-in. by 1⅞-in. ledges at the ends and by a pair of 4½-in. by 1⅞-in. cleats in the middle portion. The abutting ends of the panels were provided with 1-in. by ⅛-in. wrought-iron plates screwed on.

When the cross-section of the wall is heavy, such as in high plain concrete walls, the panels may be held in position by walings tied back to ¾-in. bent bars set in the concrete at the top of each lift.

The formwork for the outside of the top lift of the walls of a reinforced concrete reservoir at Welwyn Garden City is shown in *Fig.* 26.11. Panels were used between the counterforts, and tied together by bolts through the walings and sheeting. The bolts passed through ⅝-in. diameter cardboard tubes, which were removed with a brace and bit after the concrete had hardened.

The illustration in *Fig.* 26.12 is an example of panel construction of forms applied to the construction of a plain concrete reservoir wall.

For shuttering the upper surfaces of the pyramids in the Luton settlement tanks steel forms were used in conjunction with special angle plates at the junctions of the faces of the pyramids. Near the tops of the slopes, timber forms were used. The method of tying and strutting these forms to resist the lateral pressure of the wet concrete is shown in *Fig.* 26.13.

A type of form with aluminium alloy frames and liners is shown in *Fig.* 26.14. The ties are serrated strips of aluminium alloy concreted in, the projecting ends being snapped off after the formwork is stripped.

Fig. 26.13.

Settlement Tanks

Most circular settlement tanks are operated by inclined scraper-blades, attached to a rotating arm, which carry the precipitated material towards a central sump. For the efficient working of the tank, it is necessary that the blades are always very close to the surface of the floor without touching it, so that the surface of the floor must be accurate to line and level within very small limits (say $\frac{1}{8}$ in.). This may

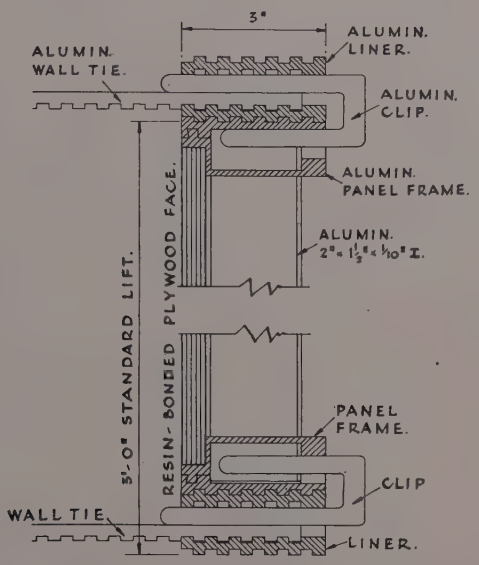


Fig. 26.14.

be accomplished by leaving the surface of the structural floor about 2 in. below the finished level with screw-in sockets cast in the surface, fixing steel templates accurately to these sockets, and laying two inches of granolithic reinforced with steel mesh. One end of the rotating arm is carried on a central pivot and the other is usually supported on a rail set on the coping of the wall. In some modern tanks

Fig. 26.15.

the metal scraper blades are replaced by swing-mounted rubber squeegees and the rotating arm is carried on pneumatic tyres running directly on the top of the outside wall. It is sometimes possible to leave the surfacing of the floor until after the rotating arm and blades are fixed and thus use them to level or help to level the floor (see *Fig.* 26.15).

In *Fig.* 26.16 a shutter is shown which was made for the construction of sixteen conical bases, 80 ft. in diameter and sloping at 30 degrees, of sludge-digestion tanks for the Birmingham Tame and Rea District Drainage Board. The concrete

Fig. 26.16.

was placed continuously, and the difficulty of placing concrete beneath sloping shuttering was avoided. The width of the shutter was between 2 ft. and 3 ft. The shutter was pivoted at the centre of the cone, and comprised an inclined flexible steel plate sliding on guide members; it was rotated by compressed air.

CHAPTER TWENTY-SEVEN

EXAMPLES OF COMPLETED WORK

A SLURRY basin at a cement works is shown in *Fig.* 27.1. Slurry, consisting of a suspension of finely-divided chalk and clay in water, is used in the manufacture of cement. The solids are kept in suspension by continuous stirring and the slurry behaves as a liquid weighing about 100 lb. per cu. ft. Some designers have assumed that the suspended matter would at once fill up any fine cracks that might develop and have taken risks with the design. The slurry, in fact, will find its way through very fine cracks.

The slurry basin in *Fig.* 27.1 was designed to contain slurry weighing 100 lb. per cu. ft. It was originally intended to taper the wall but it was necessary to accommodate a series of deep bolt-pockets in the top. The wall was designed

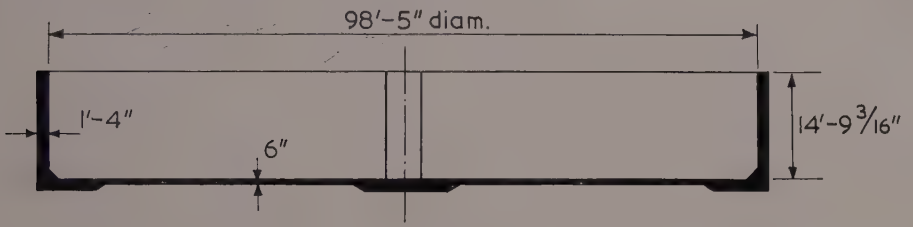


Fig. 27.1.

as a fully-restrained circular tank with a maximum tensile stress in the steel of 16,000 lb. per sq. in. both in hoop tension and in bending.

The design may be checked from the values given by the charts in Chapter 3.

$$H = 14{\cdot}3 \text{ ft.} \quad R = 49{\cdot}2 \text{ ft.} \quad T = 1{\cdot}33 \text{ ft.}$$

$$H/\sqrt{TR} = 1{\cdot}77.$$

Restraint moment $= 0{\cdot}1625wHTR$ (*Fig.* 3.13)
$= 182{,}500$ lb.-in. per foot run of wall.

$$\frac{M}{bd^2} = \frac{182{,}500}{12 \times 16^2} = 59{\cdot}3.$$

Maximum ring tension $= 0{\cdot}285wHR = 20{,}000$ lb. per foot height of wall.

$$\text{Tension on concrete alone} = \frac{20{,}000}{12 \times 16} = 104 \text{ lb. per sq. in.}$$

A smaller but similar basin is shown in *Fig.* 27.2.

Five slurry mixing tanks are further examples of the design of circular tanks. Of the five tanks, four are of 66 ft. internal diameter and one is 48 ft. in diameter. Each of these tanks is 11 ft. 6 in. deep from coping to floor level with an allowance of 6 in. freeboard. The walls are 6 in. thick for the entire height; a top projection, 8 in. deep and 4 in. wide, provides support for the segmental track plates. The floors of the tanks are of reinforced concrete 5 in. in thickness and laid on top of 2 in. of plain concrete laid on the chalk. The plain concrete was used in order

to provide a clean and firm working surface. At the junction of the walls and floor slabs 6-in. by 6-in. fillets were constructed both inside and outside the tanks. The floor of each tank is reinforced with a mesh of $\frac{1}{2}$-in. diam. bars at 18-in. centres near the top of the slab. Short $\frac{1}{2}$-in. diam. bars at 6-in. centres were laid radially in the top portion of the slab and extend inwards under the wall from the outer edge of the footing to bond the wall and floor slab together. The walls are reinforced to resist the tension caused by the pressure due to a liquid weighing 104 lb. per cubic foot and the negative moment induced at the junction of the wall and floor by the restraint which occurs at that point. (In the design of slurry tanks at Shoreham in the year 1951, Dr. Oscar Faber, assumed densities of 76 lb. per cubic foot for clay slurry and 100 lb. per cubic foot of chalk-clay slurry.)

In the 66-ft. tanks the lowest ring of hoop steel in the wall is situated 12 in. above the top of the 6-in. by 6-in. fillet. The spacings of the successive hoops of 1-in. diam. bars, counting from below, are five spaces of 5 in. each, two 6-in. spaces, one 7-in., one 8-in., and one 9-in. space. Above these eleven bars the bars are spaced at 9 in.; the number and diameters of the remaining bars are two $\frac{7}{8}$-in., one $\frac{3}{4}$-in., and two $\frac{5}{8}$-in. The hooping steel is arranged with 5-ft. staggered laps and has a cover of 2 in. of concrete measured from the outer face of the walls. The vertical bars on the outer faces are $\frac{3}{8}$-in. diam. at 12-in. centres. On the inner

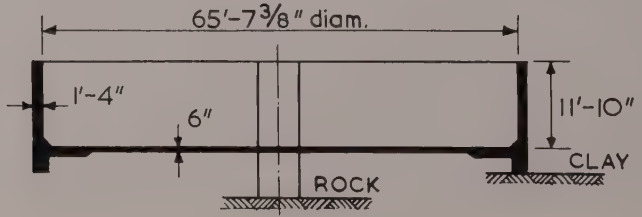


Fig. 27.2.

face the verticals are $\frac{1}{2}$-in. diam. at 6-in. centres in the lower part of the wall. At 3 ft. 6 in. above floor level the spacing of these bars is increased to 12-in. centres. The coping is reinforced with three $\frac{1}{2}$-in. diam. bars.

In the 48-ft. diameter tanks the reinforcement is similar in size and spacing to that used in the larger tanks, except in the case of the hooping bars. The spacings of these, counting from 12 in. above the top of the fillet, are two of 6 in. each and two of 7 in. for the five 1-in. diam. bars. Above these rings the bar diameters are reduced to $\frac{7}{8}$-in. and the spacings are two of 6 in. and one each of 7 in., 8 in., 9 in., 10 in. and 12 in. The two hoops at the upper part of the 6-in. wall consist of $\frac{3}{4}$-in. and $\frac{5}{8}$-in. diam. bars respectively at 12-in. centres.

During the reconstruction of another cement works two clay-slurry tanks were constructed in reinforced concrete. Each tank has an internal diameter of 66 ft. and is 12 ft. deep from coping to the top of the floor slab. The floor is 68 ft. diameter by 2 ft. thick, and is founded at 1 ft. 6 in. below the existing ground level. The slab is reinforced with a top and bottom mesh of mild steel bars, the bars in each being at 12-in. centres both ways and $\frac{3}{4}$-in. diam. The outer edge of this slab is stiffened by two 1-in. diam. bars laid circumferentially, one near the top and one near the bottom of the slab.

The wall of one of these tanks is 7 in. in thickness from top to bottom and is provided with a coping 11 in. wide, 2 in. of which is bevelled on the outer side.

The total depth of the coping is 10½ in. and it is reinforced with four ¾-in. diam. bars and $\frac{3}{16}$-in. diam. links at 6-in. centres. At the bottom of the wall there is an internal fillet 10 in. by 10 in. in section. The external fillet is 10 in. high and 5 in. wide. The junction between base-slab and wall is reinforced with two series of $\frac{7}{16}$-in. diam. bars at 6-in. centres. One set is continuous from the bottom of the slab and is bent into the outer face of the wall; the second is curved from the plane of the upper mesh in the floor into the inner wall face. The main reinforcement of the wall above the fillet is circumferential and in two rings with $\frac{5}{16}$-in. diam. verticals at 16-in. centres inside both rings and on both faces. The lower ring bars are ¾-in. diam. and the upper ⅝-in. diam. The former are spaced at 5-in. centres on each face and the bars on opposite faces are staggered. The two lower rings are also spaced at 5-in. centres, and there is a 5-in. space between the lowest ⅝-in. diam. bar and the top ¾-in. diam. ring. Above this the spacing is increased to 6 in., and near the top it is 8 in. on each face. The hoop steel is bent at right-angles at both ends and lapped 3 ft. 6 in. and 4 ft. for the ⅝-in. and ¾-in. diam. bars respectively, the laps being tied together with 3 ft. of 16 S.W.G. wire at both ends.

In the first tank constructed the wall thickness varied from 7 in. to 4 in. below the coping, but the second tank was built with a wall of uniform thickness in order to simplify the circular formwork and to permit the use of standard steel forms.

The concrete in the walls of the tank was mixed in the proportions of 6 cwt. of rapid-hardening Portland cement to ½ cu. yd. of sand and 1 cu. yd. of ¾-in. to ¼-in. shingle. In the concrete for the floor slab a proportion of 1-in. to 1½-in. shingle was allowed.

A plan and sections of a 1,000,000-gallons covered reservoir at Penrith, which is 124 ft. by 100 ft. in plan and has a depth of water of 13 ft. are shown in *Figs.* 27.3 and 27.4. The roof is supported by thirty-six 11-in. square columns, each reinforced with four ¾-in. diam. bars.

The walls are designed to span vertically between the roof and the floor, and are assumed to be fixed at the bottom and freely supported at the top. Fixity at the bottom was ensured by increasing the thickness of the concrete under the walls and providing the necessary steel in the base. The thickness of the wall varies from 8 in. at the top to 12 in. at the bottom. The arrangement of the mild steel reinforcement in the wall is shown in the illustrations. The main bars are vertical, and ½-in. diam. horizontal bars at 24-in. centres are also provided near both faces of all walls to resist temperature and shrinkage stresses. The external walls were designed to resist pressure from either the head of water in the reservoir or the earth backing acting independently, as it was considered that the backing might at some time shrink away from the wall. The inner partition is designed to withstand water pressure from either side.

For the floor of the reservoir 1 in. of concrete was first placed and steel mesh reinforcement laid thereon in continuous lengths 7 ft. wide to enable the floor to span over any hollows due to differential settlement of the ground beneath. As will be seen from the drawings the base is thickened under each column and reinforced with mesh laid in two directions (at right-angles) to distribute the load evenly on the ground. The direct tension in the base slab is resisted by mesh, with the main wires at right-angles to the length of the walls. This mesh is extended 12 ft. inwards. Beyond this point no steel is required for this purpose, as the friction between the ground and the tank bottom is sufficient to reduce the tension to a value that may be resisted by the concrete. The aggregate consisted of ¾-in.

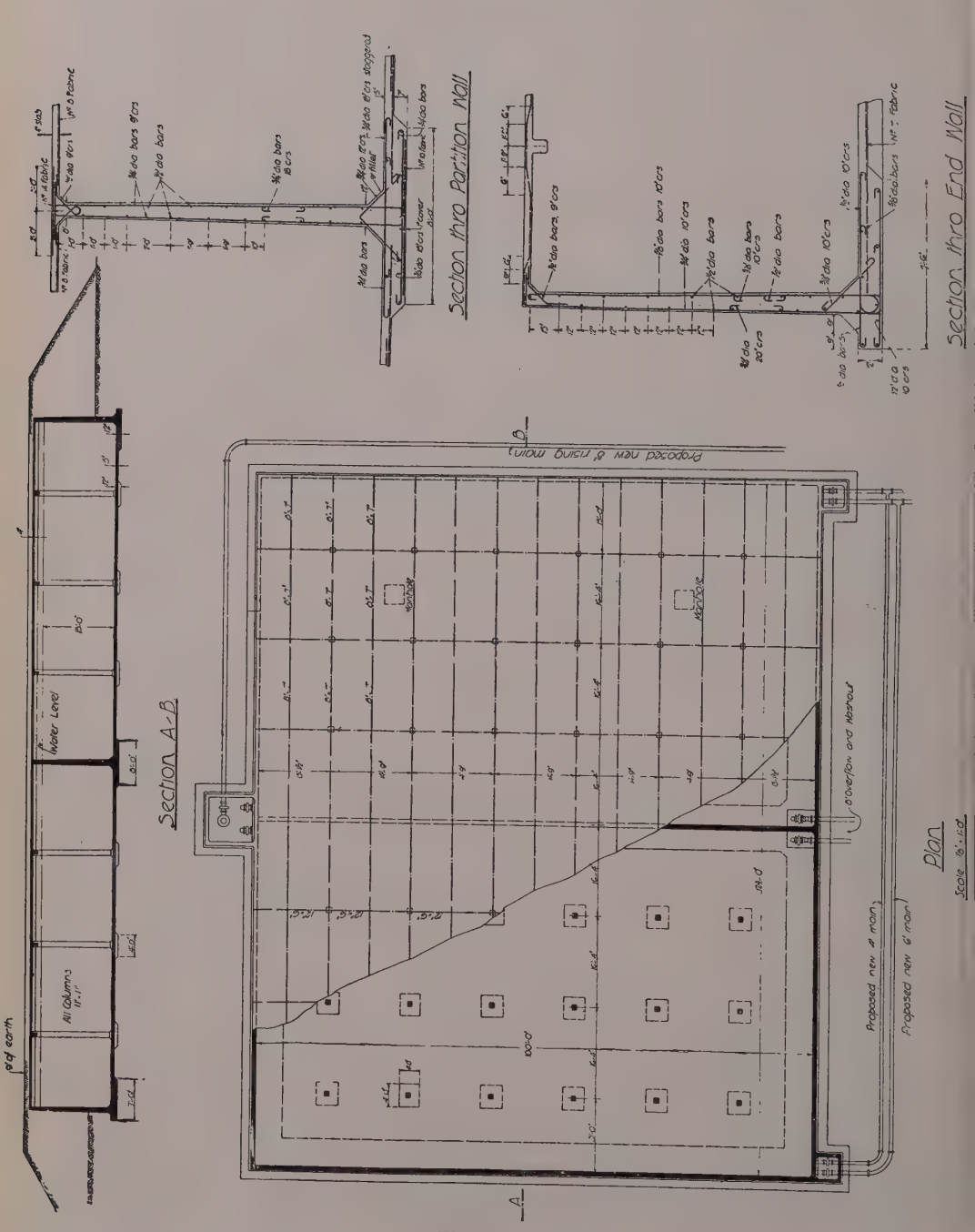
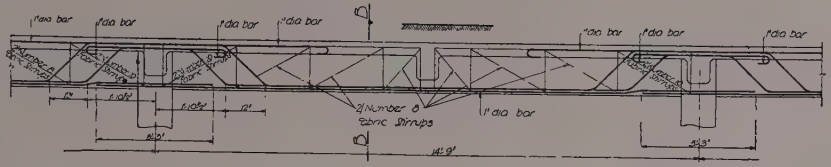
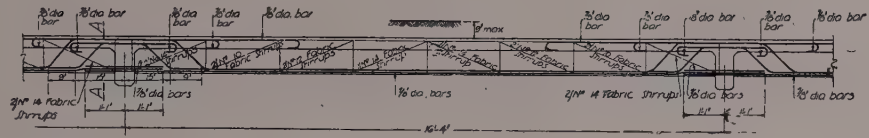

Section thro Partition Wall
Section thro End Wall
Section A-B
Plan
Scale 16 : 1 ft
Proposed new 6" rising main
Proposed new 4" main
Overflow and Washout
Inlet
Outlet
Water Level
All Columns 11'-11"
A
B

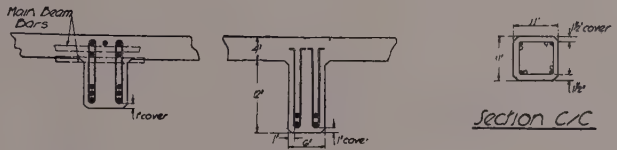
Fig. 27.3.



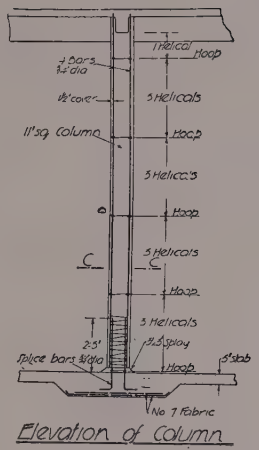
Main Beam



Secondary Beam



Section A-A Section B-B Section C-C



Elevation of Column

Fig. 27.4.

granite chippings and river sand. The proportions of the concrete were 1 : 2 : 4 in the floor and roof and 1 : 1½ : 3 in the walls and columns.

On excavating to the specified level, half the bottom was found to be composed of sandstone rock and half of sandy clay. Land drains were formed around the foundation to the walls leading to the sandstone, and by this means the water was drained away through the fissures in the rock. Under two-thirds of the columns, after excavation had been carried to the required level, the ground was

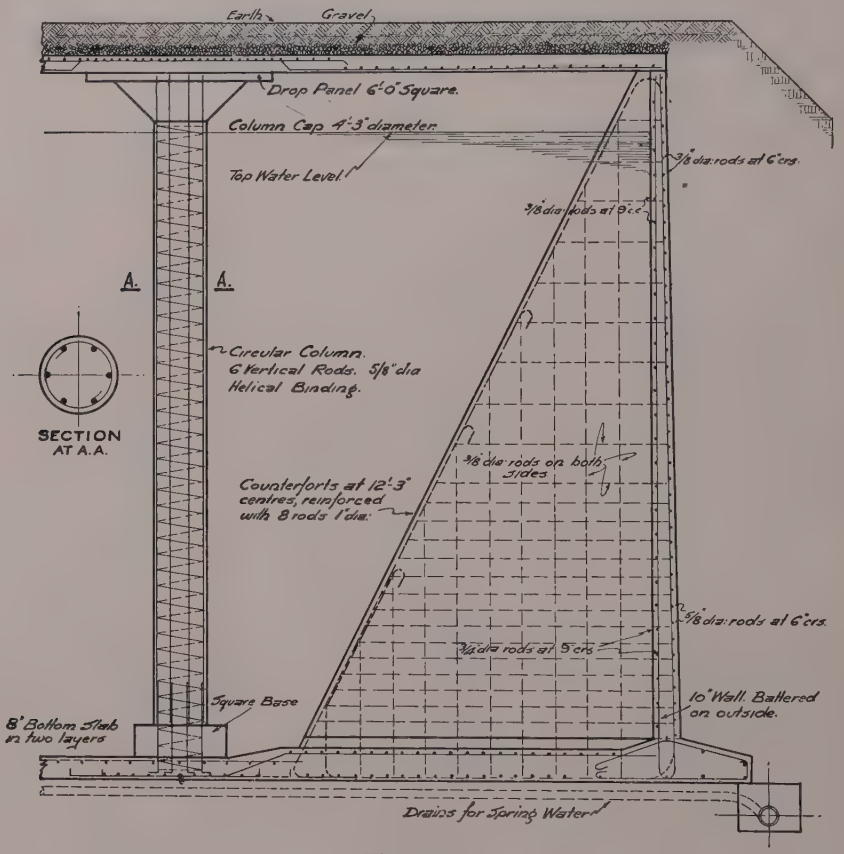

Earth
Gravel
Drop Panel 6'-0" Square.
Column Cap 4'-3" diameter.
Top Water Level
⅝" dia. rods at 6" crs.
⅝" dia. rods at 9" c.c.
A.
A.
Circular Column. 6 Vertical Rods. ⅝" dia. Helical Binding.
SECTION AT A.A.
⅝" dia. rods on both sides
Counterforts at 12'-3" centres, reinforced with 8 rods 1" dia.
⅝" dia. rods at 6" crs.
¾" dia. rods at 9" crs.
Square Base
10" Wall, Battered on outside.
8" Bottom Slab in two layers
Drains for Spring Water

Fig. 27.5.

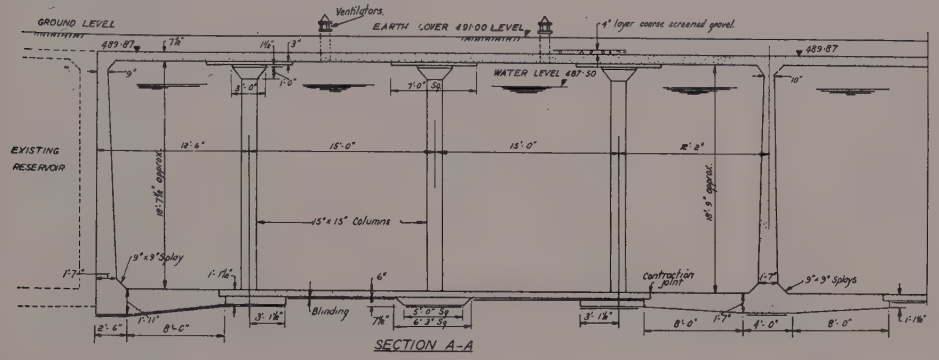
found to be in such a spongy state, due to inclement weather, that about 6 in. of plain concrete was laid.

The formwork for the walls was removed in from two to three days, depending on the weather. The sides of the beams were removed in three to four days, and props to soffits after five weeks. After completion of the work alternate compartments were filled for the purpose of testing, but there was no sign of any leakage. No waterproofing composition was used on the inside of the reservoir, but a coating of cement was applied.

A section through part of a service reservoir at Nelson covered with a flat-slab roof is shown in *Fig.* 27.5. The walls were designed to retain 22 ft. of earth with the reservoir empty and a head of 20 ft. of water when filled. A test was carried out before the embankments were filled. No asphalt or other lining was

used. The dimensions of this reservoir are: length 305 ft.; width 135 ft.; average depth of water 20 ft.; the capacity is 5,000,000 gallons.

A service reservoir with a capacity of 1,000,000 gallons for the Borough of Stafford was constructed abutting an existing reservoir, the wall of which formed



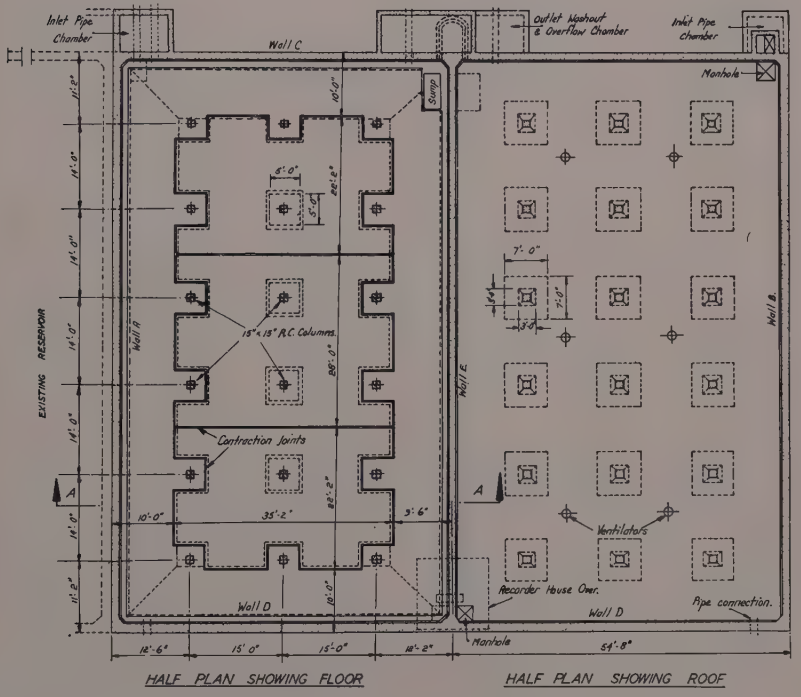
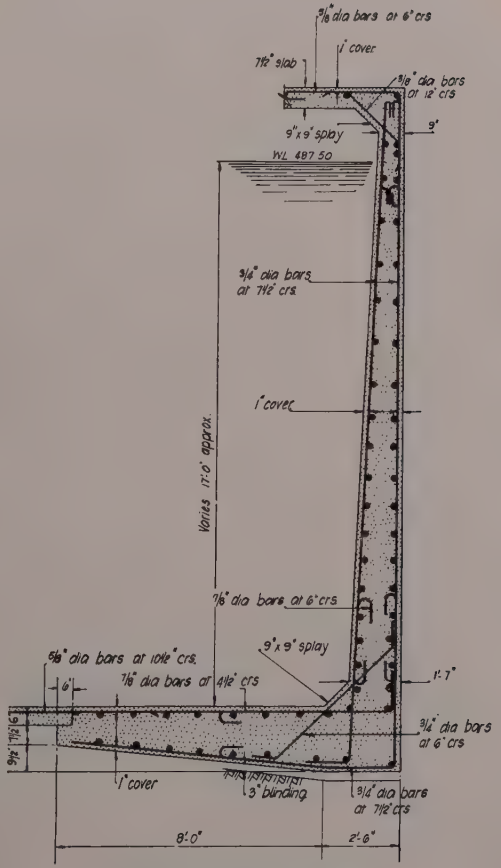
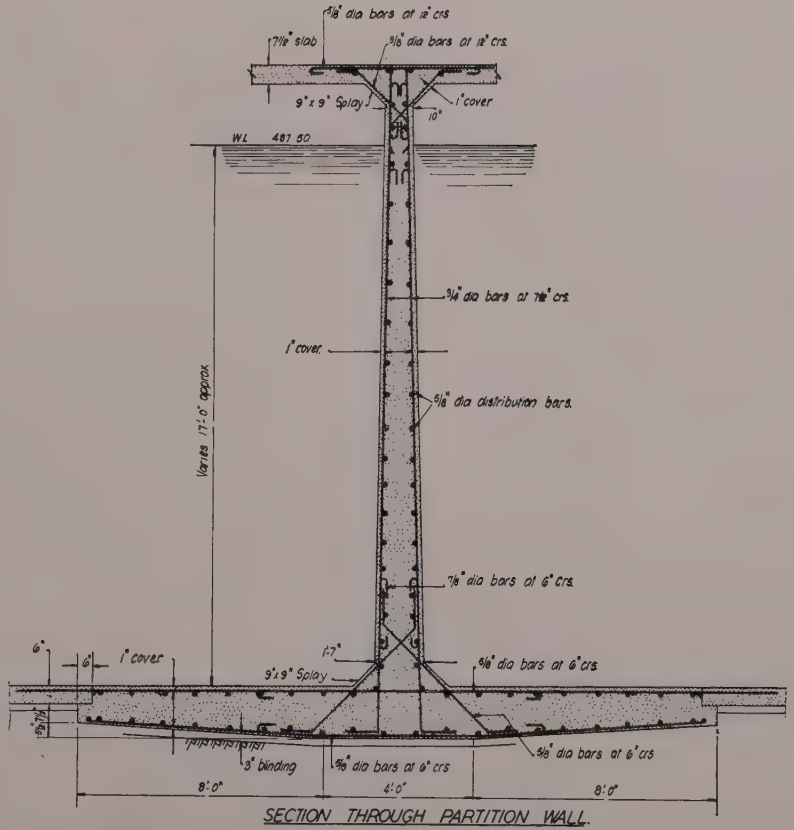


Fig. 27.6.

the outer shutter for one wall of the new work. The inside dimensions of the new reservoir, measured at floor level, are 106 ft. 2 in. by 90 ft. 2 in. by about 18 ft. 9 in. from the floor to the soffit of the roof (*Fig.* 27.6). A wall divides the reservoir into two compartments of equal capacity connected by a 15-in. diameter pipe. A pipe of the same size connects the new and existing reservoirs, and the mains are so arranged that each compartment can be used independently.



SECTION THROUGH END WALL.



SECTION THROUGH PARTITION WALL.

Fig. 27.7.

The reservoir is founded on sandstone and gravel, and the site was first covered with a 3-in. layer of 1 : 2 : 4 concrete. The floor, which was cast in six panels, is 6 in. thick and is reinforced in two directions with mild steel bars. The column bases were cast with the floor and are 5 ft. square by $7\frac{1}{2}$ in. thick below the floor slab. The walls (*Fig.* 27.7) are designed as propped cantilevers. The outer walls taper from 1 ft. 7 in. thick at the base to 9 in. at the roof, the outer face being vertical. The dividing wall tapers on both faces from 1 ft. 7 in. to 10 in. Splays,

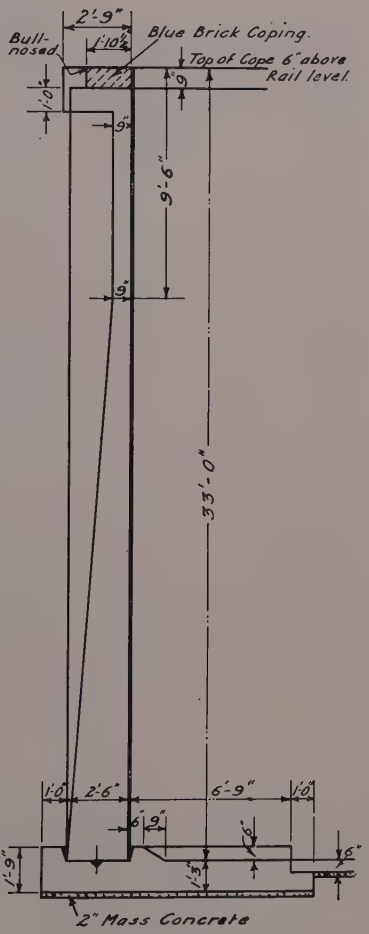


Fig. 27.8.

9 in. by 9 in., are provided at all horizontal and vertical corners. No allowance was made for support from the earth on the outer walls, and the dividing wall was designed to resist the stresses when either compartment only is full.

The roof is a flat slab $7\frac{1}{2}$ in. in thickness, with drop panels 3 in. deep by 7 ft. square supported on thirty-six columns 1 ft. 3 in. square. The roof is covered with 4 in. of screened gravel, 2 in. to $\frac{3}{4}$ in. in size, and 9 in. of soil. In addition, the roof is designed to carry an imposed load of 100 lb. per square foot. The concrete mix for all the work comprised 1 cwt. cement, 2 cu. ft. sand, and 4 cu. ft. coarse aggregate.

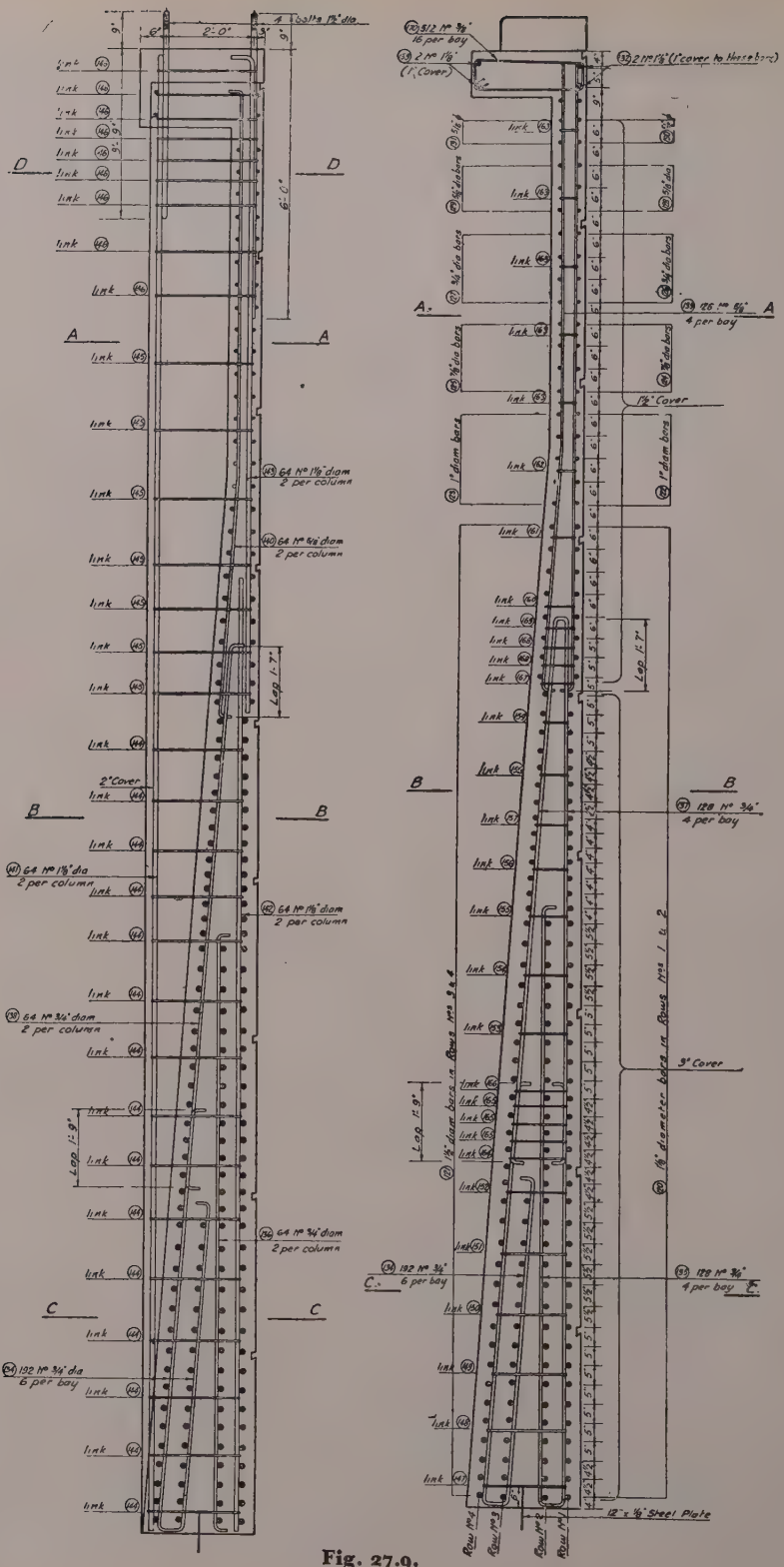

D
D
A
A
B
B
C
C
64 No 1⅛" diam 2 per column
64 No ⅞" diam 2 per column
64 No 1⅛" diam 2 per column
64 No 1⅛" diam 2 per column
64 No ¾" diam 2 per column
64 No ¾" diam 2 per column
192 No ¾" dia 6 per bay
512 No ¾" 16 per bay
2 No 1⅛" (1" Cover)
2 No 1⅛" (1" cover to Horse bars)
128 No ⅞" 4 per bay
128 No ¾" 4 per bay
128 No ¾" 4 per bay
192 No ¾" 6 per bay
1½" Cover
3" Cover
12" x ½" Steel Plate
Row No 1
Row No 2
Row No 3
Row No 4

Fig. 27.9.

Along all horizontal and vertical daywork joints in the walls, $\frac{3}{4}$-in. by $\frac{3}{4}$-in. V-shaped grooves were formed on the inside face. The grooves were later primed and filled with a sealing compound applied with hot rollers. Two coats of waterproofing solution were then applied over the joints to form a band 6 in. wide. The daywork joints in the floor are grooves $\frac{3}{4}$ in. wide by $1\frac{1}{2}$ in. deep, which were treated

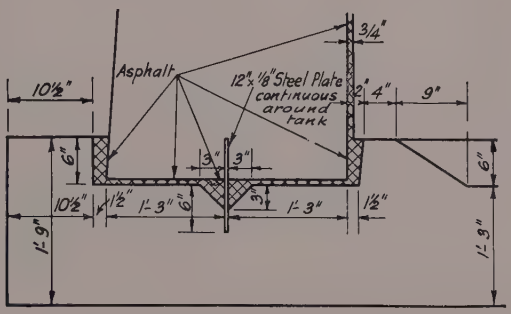


Fig. 27.10.

in a similar manner. Three external valve chambers were constructed after the completion of the main walls. The puddle flange-pipes were cast in the walls during construction.

Some unusually interesting details are shown in *Figs.* 27.8 to 27.10, from the drawings of a reinforced concrete gasholder tank 162 ft. in diameter and 33 ft. deep,

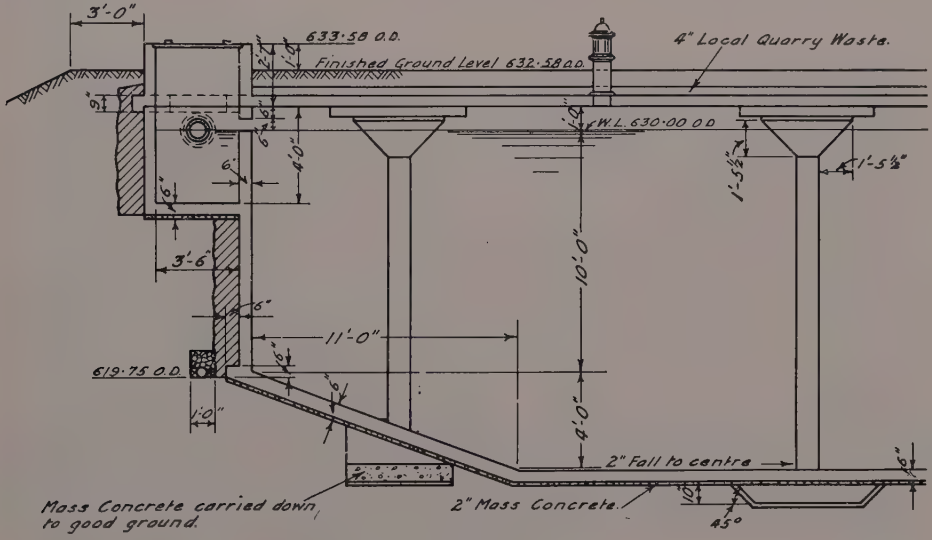


Fig. 27.11.

the walls of which are lined with $\frac{3}{4}$ in. of asphalt. The capacity of the tank is 4,250,000 gallons and the 2,500,000-cu.-ft. holder is of the spirally-guided type with four lifts. The tank is entirely below ground level, and the wall (*Fig.* 27.8) varies in thickness from 2 ft. 6 in. at the base to 9 in. at a distance of 9 ft. 6 in. below coping level. Upwards from this point the thickness is uniformly 9 in. Thirty-two piers support the stanchion bases, and the coping is 2 ft. 9 in. wide by 1 ft. 9 in. deep. The footing slab (*Fig.* 27.10) under the wall is 1 ft. 9 in. in thickness and is recessed

to a depth of 6 in. on a width of 2 ft. 9 in. to receive the wall. From the middle of this slot a 12-in. by $\frac{1}{8}$-in. steel plate, continuous around the tank, projects 6 in. into the wall and forms a joint between wall and base. The thickness of the main part of the floor is 6 in. The reinforcement for the wall is shown in *Fig.* 27.9.

Some details of a covered circular reservoir 64 ft. 3 in. in diameter, of which the bottom is partly sloping and partly horizontal, are shown in *Figs.* 27.11 to 27.13. Of these, *Fig.* 27.11 is not typical in so far as the wall is concerned: the typical wall section is that shown in *Fig.* 27.13.

A reservoir with a capacity of 1,500,000 gallons and a 1,000,000-gallon elevated tank 37 ft. 6 in. above the reservoir are shown in *Figs.* 27.14 to 27.18. They were built for the city of Glasgow and are situated to the north of the city at the highest

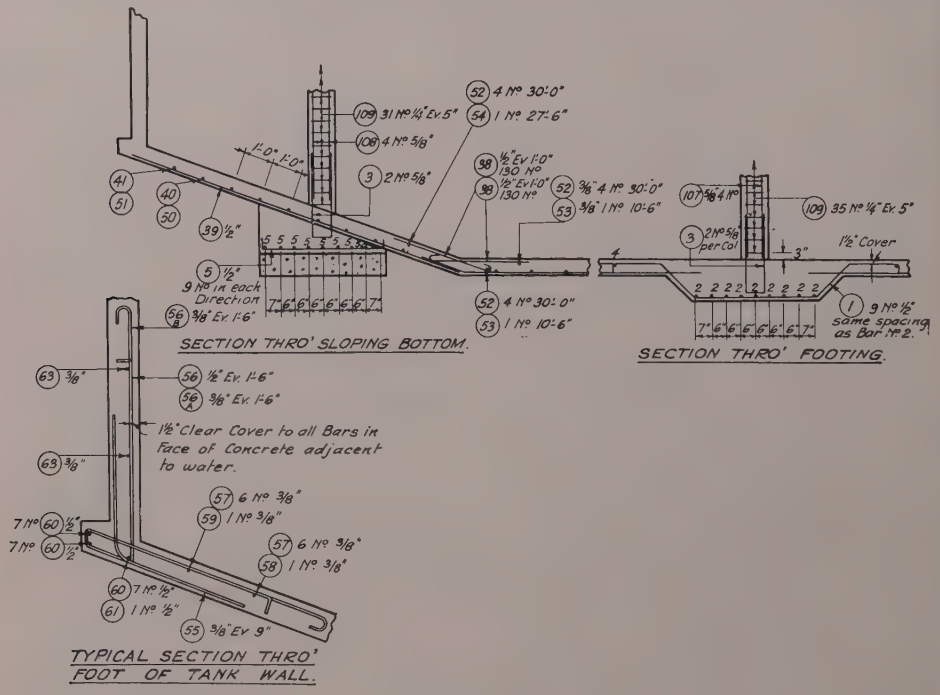

SECTION THRO' SLOPING BOTTOM.
SECTION THRO' FOOTING.
1½" Clear Cover to all Bars in Face of Concrete adjacent to water.
TYPICAL SECTION THRO' FOOT OF TANK WALL.

Fig. 27.12.

point within its boundaries. Both the reservoir and tank are divided into two equal compartments by means of twin division walls which are separated by a $\frac{1}{2}$-in. layer of bituminous sheeting. The halves are designed to act independently of each other. The elevated tank is a plain rectangular box carried on internal columns with horizontal bracing and with large streamlined outer columns of hollow construction. At two of the corners of the tank there are square towers, inside which are staircases from the roof of the reservoir to the roof of the tank; at the other corners mock towers are carried up to provide the necessary supports to the corners of the tank and to preserve the æsthetic effect. Means of access are provided to the reservoir and tank.

The floor of the reservoir was divided into nine approximately equal areas and concreted in alternate panels in numerical order as shown in *Fig.* 27.15. A V-shaped groove $\frac{1}{2}$ in. deep at the top of the slab was formed with a timber fillet nailed to

the stop-board. Before laying new concrete against set concrete, the face of the set concrete was roughened by hand picking, cleaned, damped, and coated with

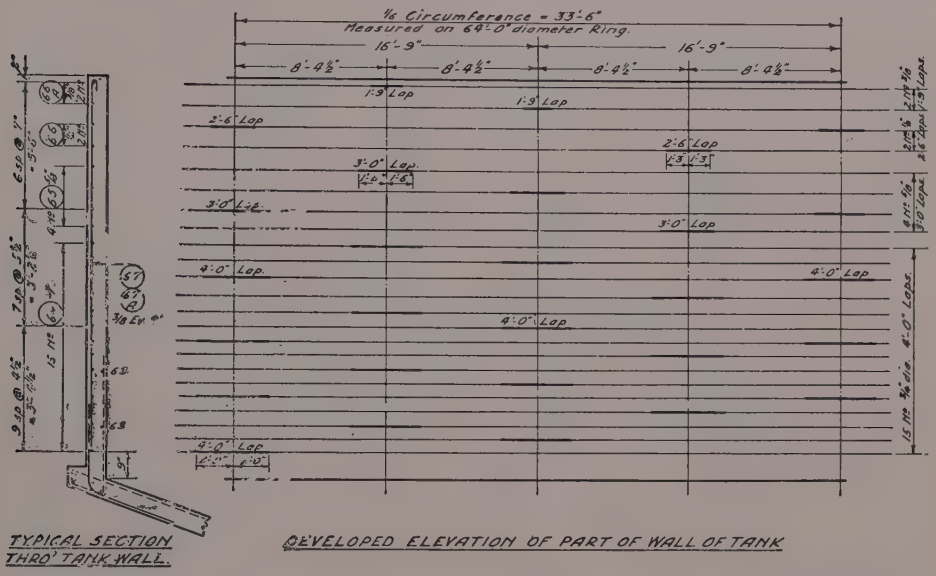


Fig. 27.13.

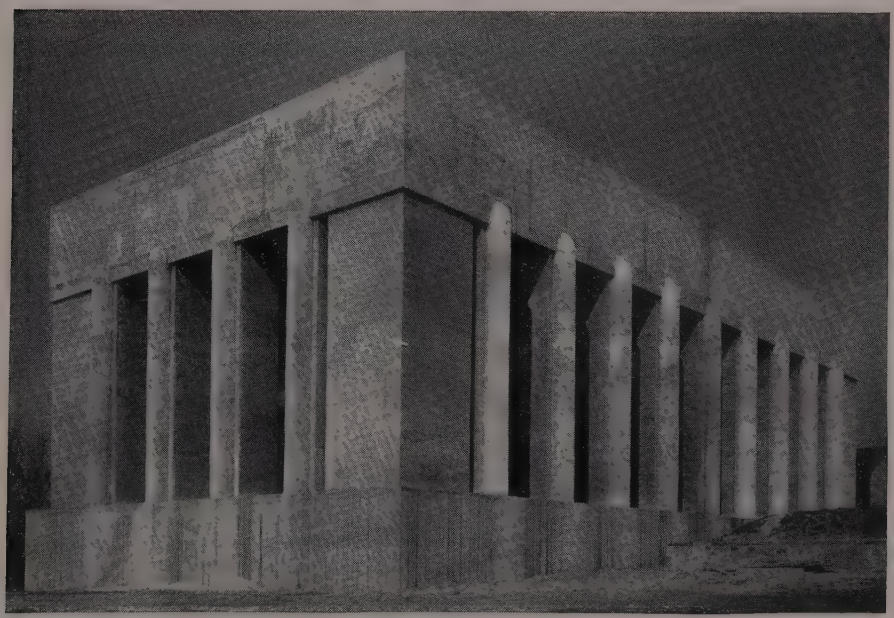
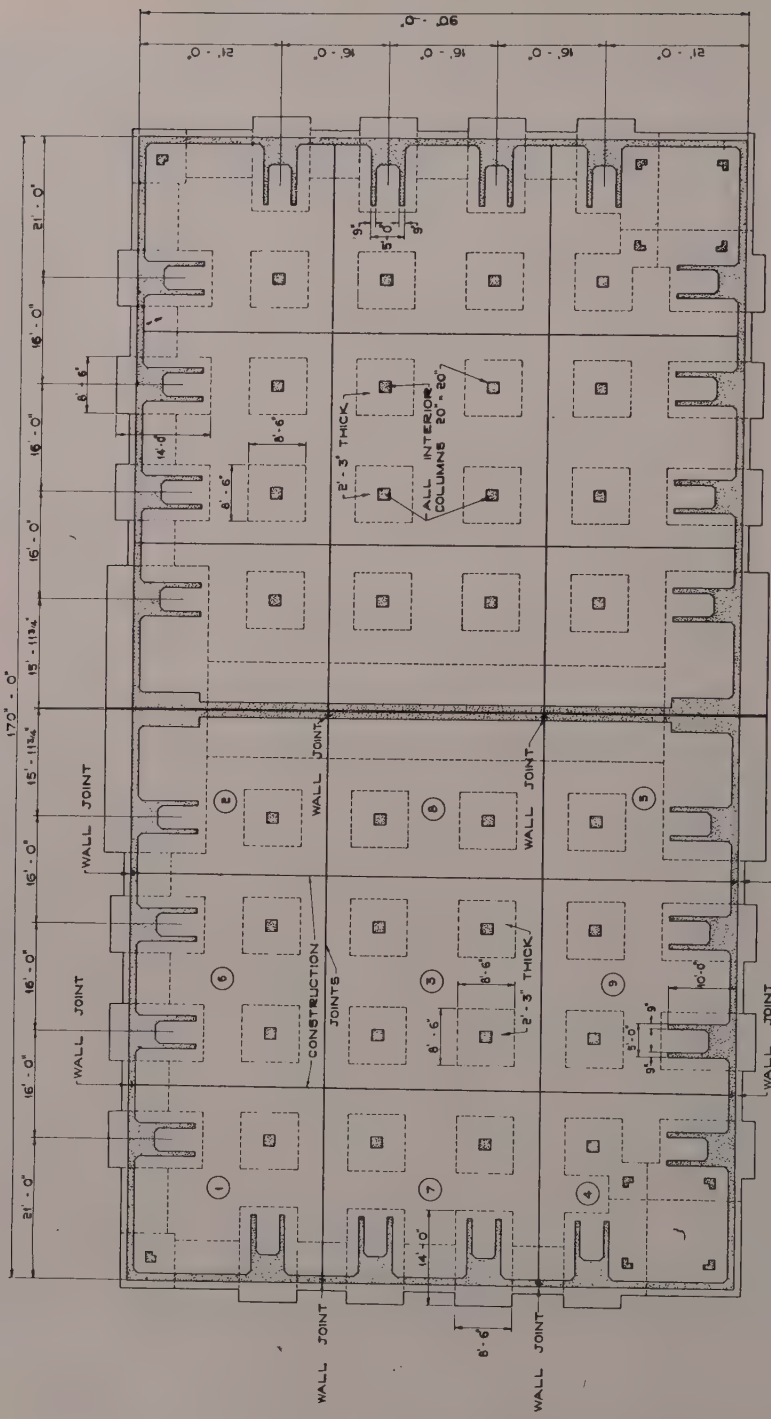


Fig. 27.14.

1 in. of stiff cement mortar immediately prior to laying the new concrete. Before sealing the V-shaped joint with the bituminous material, the joint was brushed clean of dirt and dust, first with a steel brush and finally with a soft hair brush;



PLAN OF FOUNDATIONS.

Fig. 27.15.

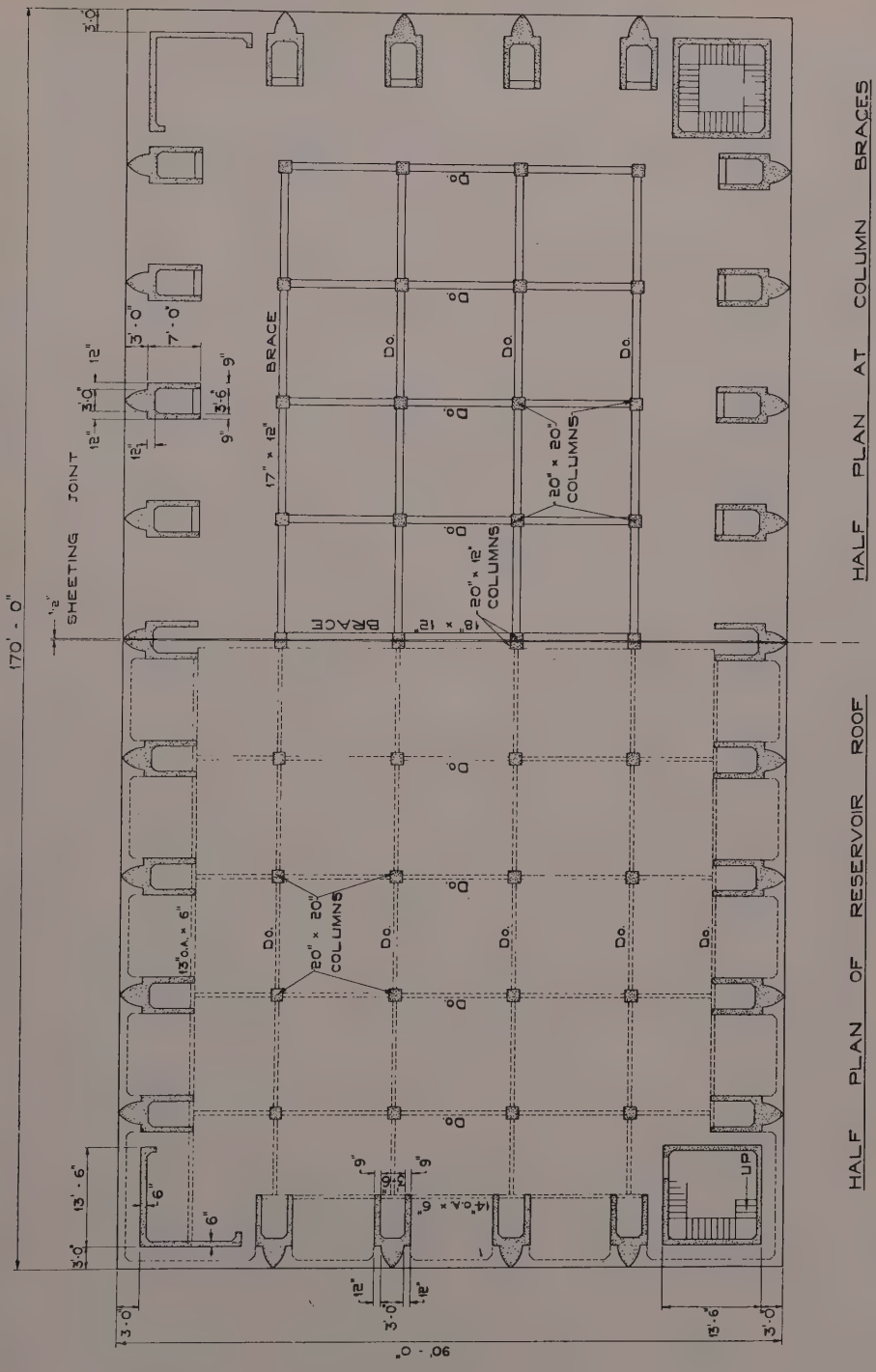

HALF PLAN OF RESERVOIR ROOF
HALF PLAN AT COLUMN BRACES
BRACE
17" × 12"
20" × 20" COLUMNS
20" × 12" COLUMNS
SHEETING JOINT
Do.
UP
170' - 0"
90' - 0"

Fig. 27.16.

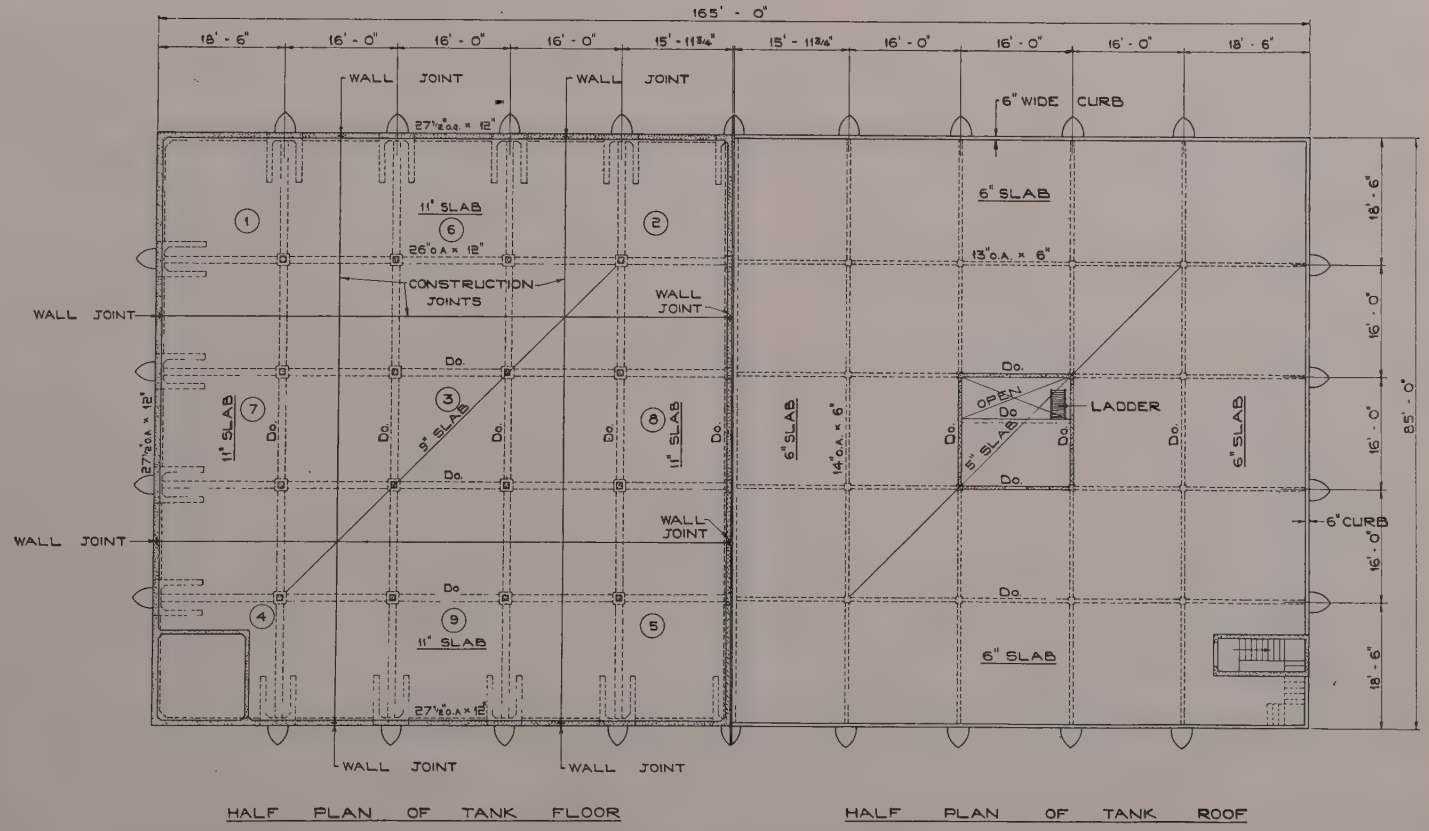
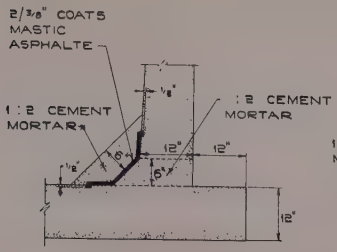
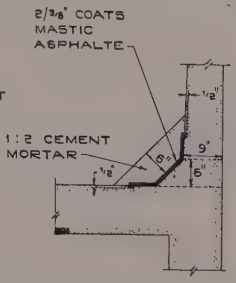

165' - 0"
18' - 6"
16' - 0"
16' - 0"
16' - 0"
15' - 11¾"
15' - 11¾"
16' - 0"
16' - 0"
16' - 0"
18' - 6"
WALL JOINT
WALL JOINT
6" WIDE CURB
27½"O.A. × 12"
11" SLAB
26"O.A × 12"
CONSTRUCTION JOINTS
WALL JOINT
6" SLAB
13"O.A. × 6"
WALL JOINT
Do.
OPEN
LADDER
9" SLAB
11" SLAB
6" SLAB
14"O.A. × 6"
5" SLAB
6" CURB
WALL JOINT
27½"O.A. × 12"
11" SLAB
18' - 9"
16' - 0"
16' - 0"
16' - 0"
18' - 9"
85' - 0"
WALL JOINT
WALL JOINT
HALF PLAN OF TANK FLOOR
HALF PLAN OF TANK ROOF

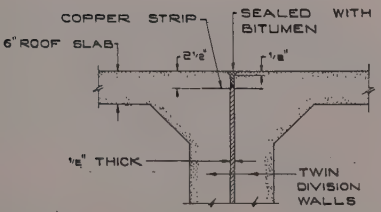
Fig. 27.17.



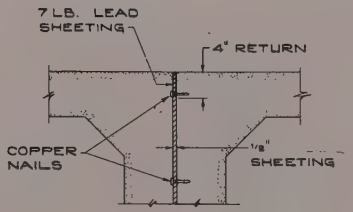
JOINT AT JUNCTION OF RESERVOIR FLOOR & WALL



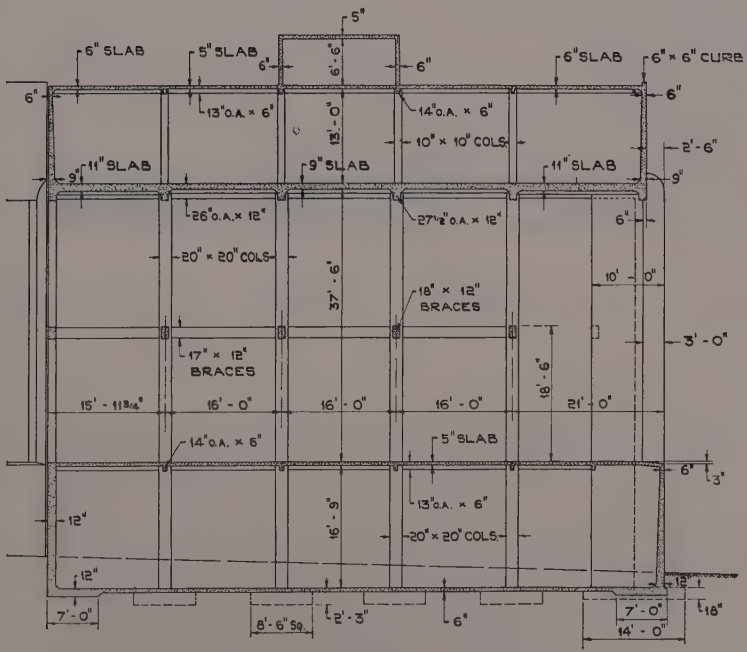
JOINT AT JUNCTION OF TANK FLOOR & WALL



EXPANSION JOINT IN TANK ROOF
JOINT IN RESERVOIR ROOF SIMILAR



SECTIONAL PLAN OF EXPANSION JOINT IN TANK WALL



HALF LONGITUDINAL SECTION

Fig. 27.18.

the joints were then cleaned of fine dust with compressed air and thoroughly dried with a blow-lamp. Before the concrete had cooled, a thin coat of bituminous material about $\frac{1}{16}$ in. to $\frac{1}{10}$ in. thick was applied to the surface of the groove with a warmed metal tool, working the coat firmly against the concrete so as to ensure adhesion. After the primary coat had been applied, the groove was filled with bituminous material with a warmed shaped-metal jointing tool; the filling was pressed firmly against the primary coat to ensure adhesion and watertightness. On completion, the bituminous filling, together with the concrete for a width of $4\frac{1}{2}$ in. on each side of the joint, was painted with two coats of bituminous solution. A similar joint was used in the walls of the reservoir and the tank and in the tank floor, with the additional provision of a perforated copper strip.

The walls of the reservoir and the tank were divided into panels by vertical

Fig. 27.19.

joints midway between the main outer columns and in line with the joints in the floor; concreting was again carried out on the alternate panel principle and the wall was concreted in one operation for its entire height.

At the junction of the floor and the wall, in both the reservoir and the tank, the bottom 6 in. of the wall and the 6-in. by 6-in. splay are constructed of 1 : 2 cement mortar. The concrete of the floor was thoroughly cleaned and roughened where it was to come into contact with the wall and splay. The 1 : 2 cement mortar, mixed fairly stiff, was then placed and was followed immediately by the structural concrete, which was thoroughly rammed into the mortar. Further protection against leakage at this point is provided by the $\frac{3}{4}$-in. mastic-asphalt covering to the splay, the asphalt in turn being protected with a 6-in. cover of 1 : 2 mortar. The main joint dividing the structure into equal compartments was formed by $\frac{1}{2}$-in. thick bituminous sheeting and on the exposed faces of the elevation the sheeting was wrapped with 7-lb. sheet lead.

The supply and discharge pipes to the elevated tank are accommodated in the hollow outside main column. Where the pipes pass through the walls or floor special castings were made. These castings are double-flanged, with a special flange

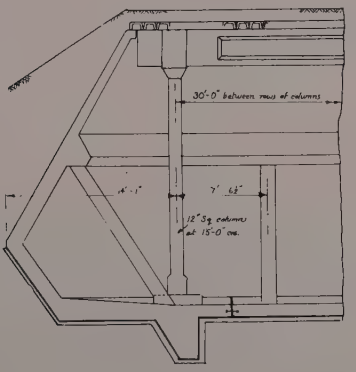


Fig. 27.20.

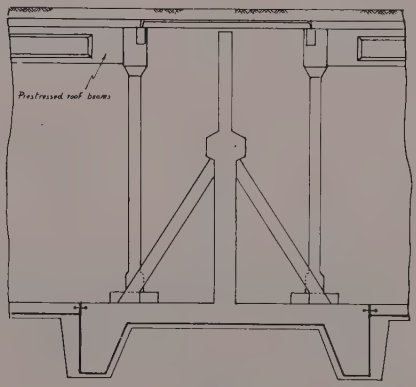


Fig. 27.21.

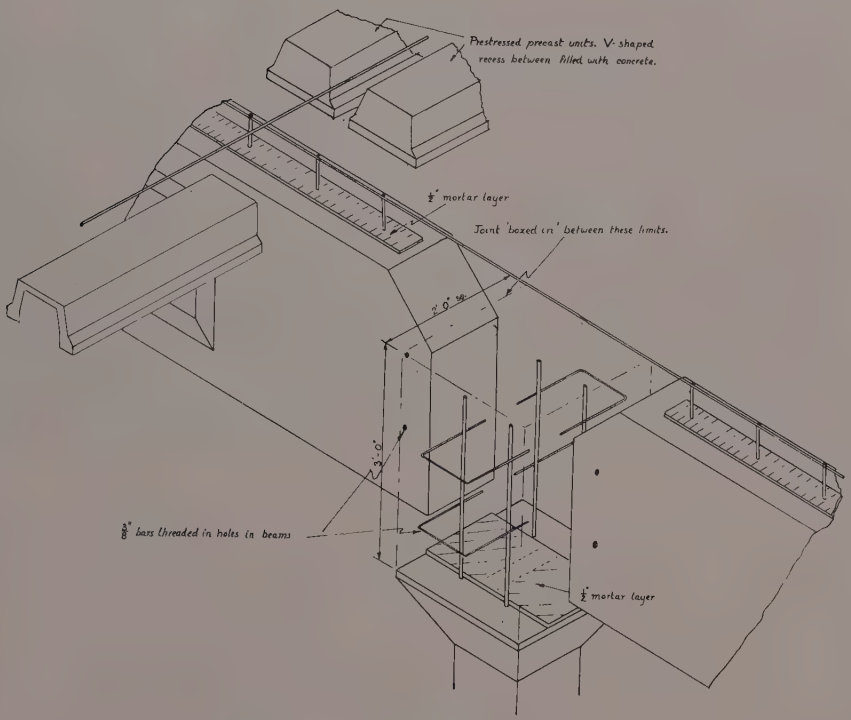

Prestressed precast units. V-shaped recess between filled with concrete.
½" mortar layer
Joint 'boxed in' between these limits.
2'-0"
3'-0"
⅞" bars threaded in holes in beams
½" mortar layer

Fig. 27.22.

cast on at the centre; they were placed in position and concreted in with the work, and the space between the flange and face of the concrete was caulked with mastic asphalt. When the reservoir and tank were filled with water these connections proved very satisfactory and showed no sign of leakage.

Details of a covered reservoir of 25,000,000 gallons capacity for the Colne

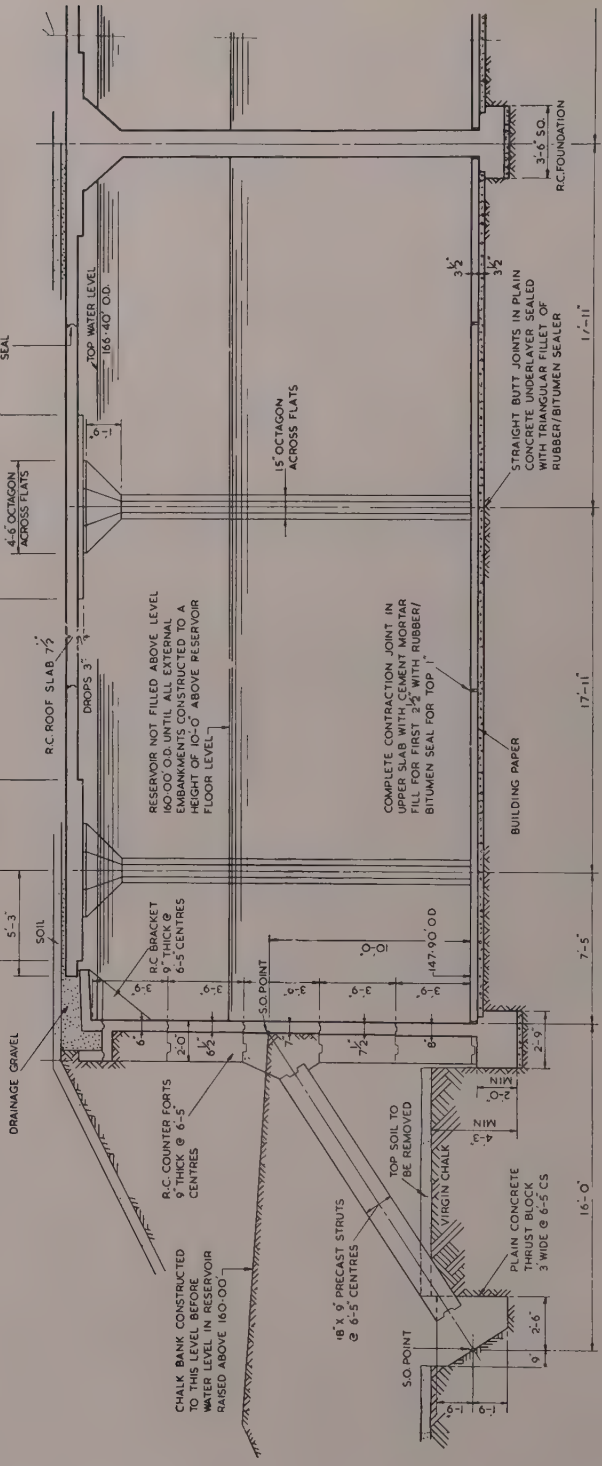

PARTIAL CONTRACTION JOINT IN ROOF SLAB WITH RUBBER/BITUMEN SEAL
TOP WATER LEVEL 166·40 O.D.
9'-0" S.O. DROPS
4'-6" OCTAGON ACROSS FLATS
15' OCTAGON ACROSS FLATS
R.C. ROOF SLAB 7"
DROPS 3"
RESERVOIR NOT FILLED ABOVE LEVEL 160·00 O.D. UNTIL ALL EXTERNAL EMBANKMENTS CONSTRUCTED TO A HEIGHT OF 10'-0" ABOVE RESERVOIR FLOOR LEVEL
COMPLETE CONTRACTION JOINT IN UPPER SLAB WITH CEMENT MORTAR FILL FOR FIRST 2'-9" WITH RUBBER/ BITUMEN SEAL FOR TOP 1"
BUILDING PAPER
STRAIGHT BUTT JOINTS IN PLAIN CONCRETE UNDERLAYER SEALED WITH TRIANGULAR FILLET OF RUBBER/BITUMEN SEALER
R.C. FOUNDATION
3'-6" SQ.
17'-11"
1'-11"
4'-6"
5'-3"
4'-6"
SOIL
R.C. BRACKET 9" THICK @ 6'-5 CENTRES
S.O. POINT
147·90 O.D.
7'-5"
2'-9"
DRAINAGE GRAVEL
R.C. COUNTER FORTS 9" THICK @ 6'-5 CENTRES
TOP SOIL TO BE REMOVED
VIRGIN CHALK
MIN
CHALK BANK CONSTRUCTED TO THIS LEVEL BEFORE WATER LEVEL IN RESERVOIR RAISED ABOVE 160·00
16" X 9" PRECAST STRUTS @ 6'-5 CENTRES
S.O. POINT
PLAIN CONCRETE THRUST BLOCK 3 WIDE @ 6'-5 CS
16'-0"
2'-6"

Fig. 27.23.

Valley Water Company are shown in *Figs.* 27.19 to 27.22. The average depth of water is 21 ft. The floor is 12 in. in thickness, and is reinforced in both directions at the top and bottom; the top reinforcement is twisted square bars. The floor was cast in alternate bays of 30 ft. by 15 ft. with expansion joints at intervals not exceeding 100 ft.

The walls are of an unusual shape. A ' wall unit ' comprises a base-slab 14 ft. wide incorporating a ' floor beam ' 4 ft. 6 in. deep, a lower section of wall 6 ft. high sloping outwards at 60 deg. and bearing against the excavated face, and an upper section of wall 20 ft. high sloping inwards at 60 deg. and incorporating a beam about mid-height. The floor and wall beams are connected by sloping reinforced concrete members at 15-ft. centres. Expansion joints are at invervals not

Fig. 27.24.

exceeding 90 ft. and there is a continuous expansion joint between the base of the wall and the floor.

The wall is designed to resist the full load of water without embankments, the the outward hydrostatic thrust being resisted by the lower sloping part of the wall and the floor beam. The weight of earth on the top inwardly-sloping section of the wall is slightly greater than the outward force due to the water pressure, and therefore the inner face of the wall will always be in compression.

The total length of the perimeter wall is more than 2,000 ft., and therefore it was economical to provide special shuttering. The wall was constructed in sections 30 ft. long, each being concreted in four lifts. The first lift comprised the base slab and floor beam, and included the building-in of the raking struts which were precast with reinforcement protruding at each end. The second lift was the lower outwardly-sloping part, followed by the inwardly-sloping part up to and including the wall beam. The final stage was the top part above the wall beam.

The dividing wall is vertical, and at mid-height is strengthened by a beam tied to the floor beams on each side by reinforced concrete members at 15-ft. centres. As in the case of the perimeter wall, the raking struts were precast and built into the base slab.

The roof is formed almost entirely of precast members. The column bases were cast insitu, and precast columns 12 in. square were erected on these bases. The main roof beams are prestressed precast members 30 ft. long. The roof comprises prestressed precast trough-shaped units 15 ft. long supported by the beams. The gaps between the sloping sides of the roof units were filled with concrete which incorporates reinforcement over the top of the main beams to produce continuity. All the prestressed roof members were designed so that their bottom surfaces are in compression under the dead load. The roof is waterproofed with three layers of bituminous emulsion reinforced with two layers of glass fibre. Soil 12 in. deep is spread over the top of the roof.

In *Figs.* 27.23 to 27.28 details are given of a service reservoir of 5,000,000 gallons capacity constructed for the Cambridge Waterworks Company. The average depth

Fig. 27.25.

is 18 ft. The roof is of flat-slab construction. The edge of the roof is not connected to the walls, there being a gap of $1\frac{1}{2}$ in. filled with a rubber-bitumen sealer. There are two complete expansion joints in both directions passing through the floor, walls, and roof, thus dividing the structure into nine separate sections.

The walls span between counterforts 2 ft. by 9 in. in cross-section, generally at 6 ft. 5 in. centres. The counterforts are propped at mid-height by struts inclined at an angle of 35 deg. to the horizontal. The struts were precast and placed in position before the thrust-blocks and counterforts were concreted. The floor is in two layers, both $3\frac{1}{2}$ in. in thickness. The lower layer is unreinforced and the upper layer is reinforced with a mesh of bars placed centrally in the slab at 10-in. intervals. Both slabs were concreted in panels 17 ft. 11 in. square with straight joints staggered between the two slabs.

The amount of concrete placed in any one operation was strictly controlled. In the case of the walls this was limited to 19 ft. 3 in. horizontally by 3 ft. 9 in. vertically. An interval of fourteen days was allowed before the intermediate panels were cast. The partial contraction joints necessitated by this procedure were pointed with rubber-bitumen seals. To prevent leakage by the water by-passing the rubber-bitumen seals, the internal face of the concrete was painted with a bituminous primer followed by two coats of bituminous paint. All the concrete

Fig. 27.26.

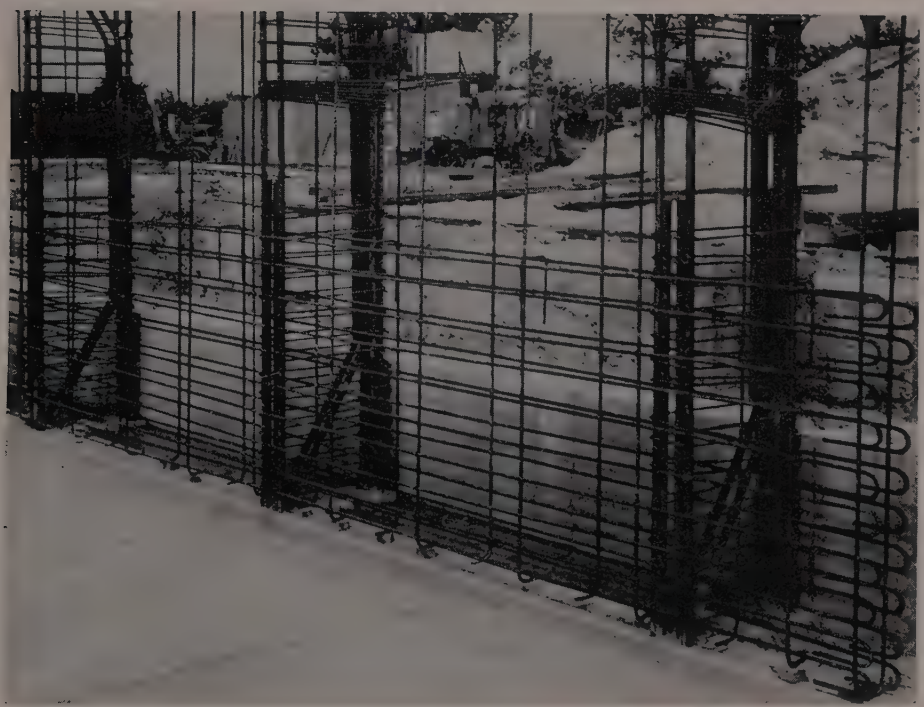
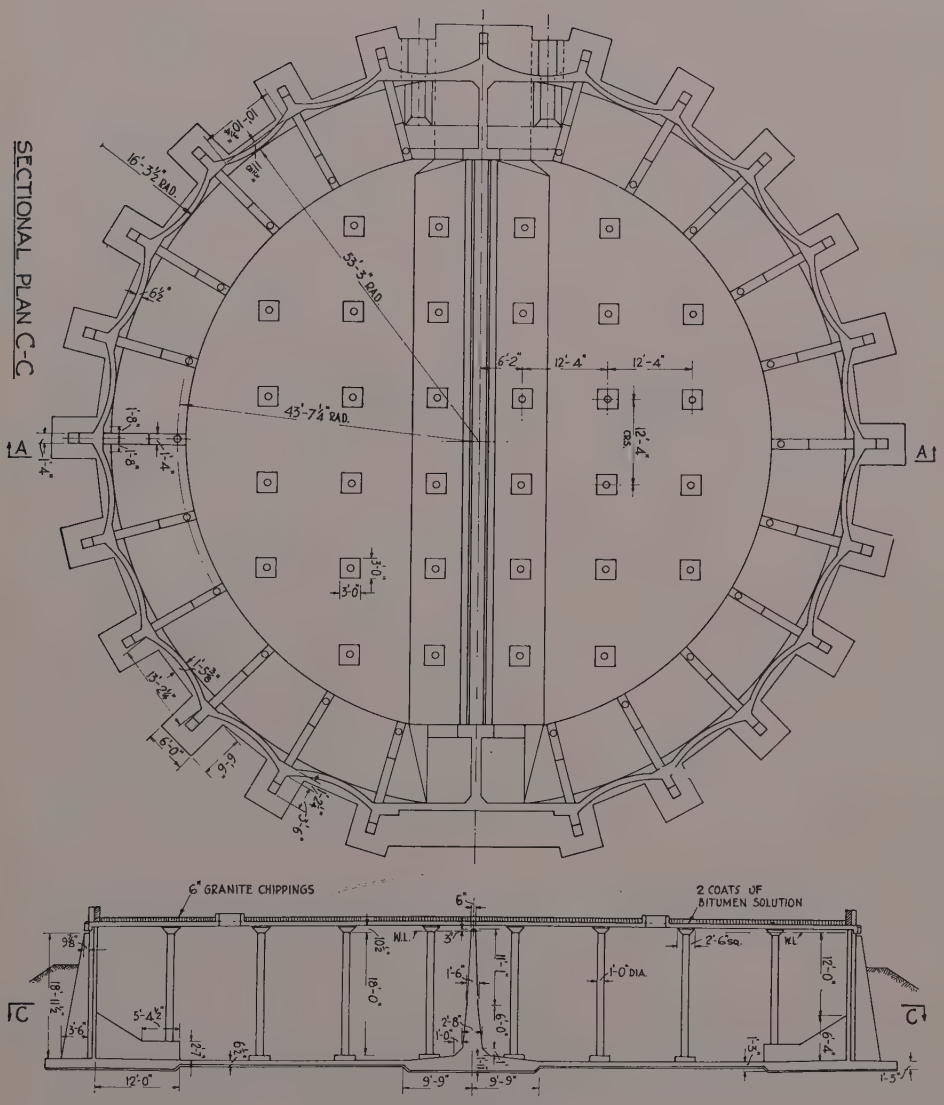


Fig. 27.27.

Fig. 27.28.



SECTION A-A

Fig. 27.29.

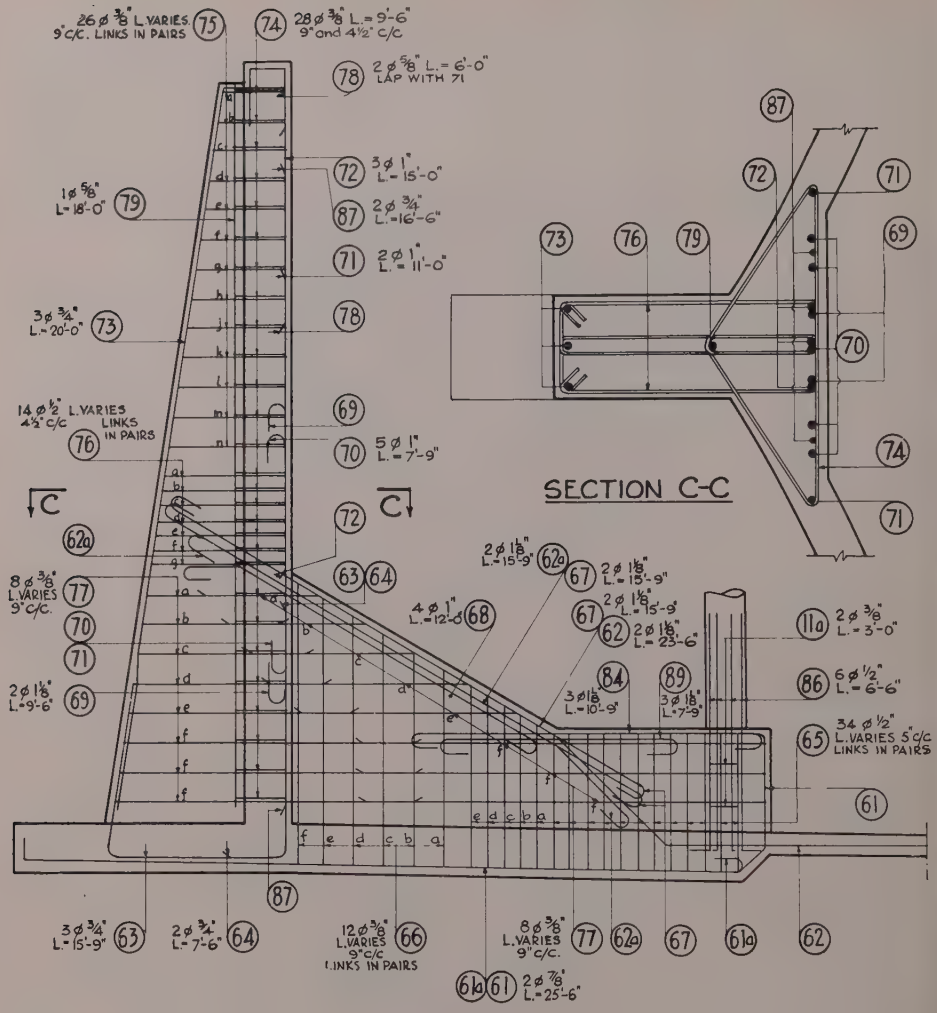

26 ø 3/8" L. VARIES 9" C/C. LINKS IN PAIRS
28 ø 3/8" L.= 9'-6" 9" and 4½" C/C
2 ø 5/8" L.= 6'-0" LAP WITH 71
3 ø 1" L.= 15'-0"
2 ø 3/4" L.= 16'-6"
2 ø 1" L.= 11'-0"
1 ø 5/8" L.= 18'-0"
3 ø 3/4" L.= 20'-0"
14 ø ½" L. VARIES 4½" C/C LINKS IN PAIRS
5 ø 1" L.= 7'-9"
SECTION C-C
8 ø 3/8" L. VARIES 9" C/C.
2 ø 1 1/8" L.= 9'-6"
4 ø 1" L.= 12'-0"
2 ø 1 1/8" L.= 15'-9"
2 ø 1 1/8" L.= 15'-9"
2 ø 1 1/8" L.= 23'-6"
3 ø 1 1/8" L.= 10'-9"
3 ø 1 1/8" L.= 7'-9"
2 ø 3/8" L.= 3'-0"
6 ø ½" L.= 6'-6"
34 ø ½" L. VARIES 5" C/C LINKS IN PAIRS
3 ø 3/4" L.= 15'-9"
2 ø 3/4" L.= 7'-6"
12 ø 3/8" L. VARIES 9" C/C LINKS IN PAIRS
8 ø 3/8" L. VARIES 9" C/C.
2 ø 7/8" L.= 25'-6"

Fig. 27.30.

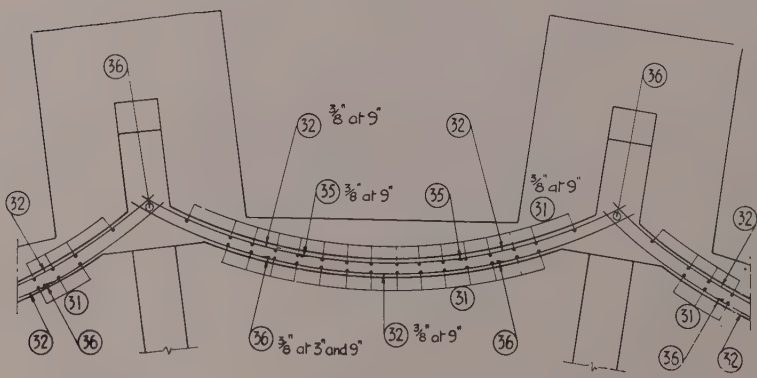


Fig. 27.31.

was compacted by vibration, and in addition the concrete in the walls and columns was agitated by sword-type spatulas 6 ft. long and 2 in. by $\frac{3}{8}$ in. in cross-section.

Details of a service reservoir at Leicester to contain 1,000,000 gallons are given in *Figs.* 27.29 to 27.32. The wall panels are segmental arches, and the primary

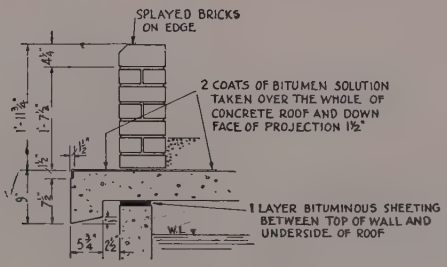

SPLAYED BRICKS ON EDGE
2 COATS OF BITUMEN SOLUTION TAKEN OVER THE WHOLE OF CONCRETE ROOF AND DOWN FACE OF PROJECTION 1½"
1 LAYER BITUMINOUS SHEETING BETWEEN TOP OF WALL AND UNDERSIDE OF ROOF
W.L.

Fig. 27.32.

stress in them is compression only. Secondary moments may be caused by shrinkage (if any) and displacement of the crown due to direct compression in the panel and outward deflection of the counterforts. The counterforts in this reservoir are structurally independent of the roof and are in tension on the water face. The division wall is a tapering cantilevered slab and the roof is of flat-slab construction.

Covering Existing Reservoirs

At the Erdington service reservoir of the Birmingham Corporation, the precast slab-and-beam roof is supported on 12-in. by 12-in. precast columns spaced at 13-ft. centres both ways. This reservoir is oval in plan, with a major axis of 400 ft. and a minor axis 300 ft. long; it is 24 ft. deep from foundation level to the top of the roof, and the latter is designed to carry 1 ft. of earth and a superload of 1 cwt. per square foot. The depth of water is 20 ft., and the plain concrete walls taper from 2 ft. 3 in. thick at the top to 6 ft. at the bottom. A facing of blue Staffordshire bricks, set in 1 : 1½ Portland cement mortar with a ½-in. coat of asphalt between the concrete and brickwork, forms the inner surface of the wall. The 12-in. concrete floor is covered with a ½-in. layer of asphalt and slopes at 1 in 200 towards a central drainage channel. So that all the columns should be the same length, the 4-ft. square footings were brought up to a level which compensated for the fall in the floor. Each footing has a rectangular socket to receive the spigot cast on the end of the column.

The precast reinforced concrete columns are 12 in. square, each having an extended base designed to provide an ample bearing on the footing and also having an extended head for the support of the beams. The longitudinal reinforcing bars of the columns were left projecting about 15 in. above the concrete, in order that the projecting hooked ends of the main reinforcing bars of the beams might be looped over them and so link together the two types of structural members.

The beams were precast to a uniform length of 12 ft., with the exception of the shorter beams connecting the marginal with the adjacent interior columns. As the spacing of the columns is 13 ft. from centre to centre, there was a distance of 12 in. between the ends of adjacent beams to provide for linking the reinforcing bars, the intervening space being afterwards filled in with concrete. The beams

Fig. 27.33.

Fig. 27.34.

are 16 in. deep by 16 in. wide at the top and are shaped off to 8 in. wide at the bottom so as to form an approximately T-shaped cross-section.

Upon the beams are carried slabs of trough section precast in widths of 24 in. and with a length a few inches less than that of the beam spacing. The slabs are 3 in. thick, each having two 6-in. projecting ribs which form the trough-shaped section, and the edges above the ribs are chamfered off to a depth of 7 in. When assembled, the slabs are placed with the trough inverted so that the projecting ribs come together and form a series of beams below the slabs, while the chamfered edges of continguous slabs form a series of V-shaped grooves, the latter being filled in with concrete. The spaces between the ends of the slabs are similarly filled with concrete, making a monolithic connection between all elements of the roof covering. This simple method of slab construction resulted in a great saving of time and labour, as it eliminated the necessity for moulding and erecting separate beams or ribs for stiffening the slabs.

In *Fig.* 27.33 a view is shown which was taken during the construction of a reinforced concrete reservoir cover, having an area of 3,100 sq. yd., and carried on precast beams and columns, at Bridlington.

Some circular reservoirs have recently been covered with domes of aluminium alloy of the Hamman type. These weigh only a few pounds per squre foot and are easily carried on a reinforced concrete thrust-ring. Such a dome of 150 ft. diameter is shown in *Fig.* 27.34. The method of construction is applicable for tanks of up to 500 ft. in diameter.

CHAPTER TWENTY-EIGHT

RESTRAINT IN PRESTRESSED CIRCULAR TANKS

At the outset of any consideration of the prestressing of circular tanks there is a striking anomaly. Theory indicates that the primary requirement must be the provision of horizontal prestressed hoops to eliminate the hoop tension in the concrete and that vertical prestressing is only of secondary importance. In practice any leakage that takes place in existing circular tanks constructed of normal reinforced concrete (assuming that they are reasonably well designed, specified and built) always occurs at horizontal joints and never through vertical cracks or vertical construction joints. From this it would appear that vertical prestressing is more important than prestressed hooping.

Some tanks have been constructed with a double system of inclined helical bars. This should greatly reduce the local temporary stresses during tensioning, but will not affect the general argument which follows and which is based on the use of horizontal and vertical reinforcement.

Wall of Uniform Thickness with Slip Joint

The simplest case is that of a wall of uniform thickness provided with an efficient frictionless sliding joint at the base (assuming that such a thing exists) and provided with hooping in horizontal planes. This case is shown in *Fig.* 28.1. AB is the original position of the wall before stressing and *ab* the designed position when the tank is full, the wall having moved inward an amount A*a* such that it is in circumferential compression when the tank is full. Assuming that the amount of

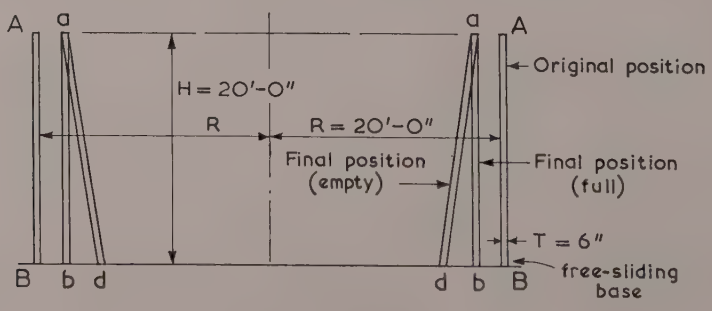


Fig. 28.1

circumferential prestress increases in proportion to the depth, the wall will move inwards to some position *ad* when the tank is empty.

If the depth H is 20 ft., the radius R is 20 ft. and the wall thickness T is 6 in., the water pressure at the base of the wall is 1,250 lb. per sq. ft. and the net hoop tension is 1,250 × 20 or 25,000 lb. per foot of height of the wall. If the hoop compression in the concrete when the wall is in position *ab* is 100 lb. per sq. in., the hoop tension in the steel at the base of the wall is 25,000 plus (6 × 12 × 100) = 25,000 + 7,200 or 32,200 lb. per foot height of wall. If the reinforcement is

stressed to 60,000 lb. per sq. in. under these conditions the area required is $\frac{32{,}200}{60{,}000}$ or 0·537 sq. in. of steel per foot height of wall. If the base of the wall moves inward a distance bd when the tank is emptied, the concrete stress rises by $\frac{bd}{R} \times E_c$ and the steel stress falls by $\frac{bd}{R} \times E_s$. Since the net hoop tension when the tank is empty is nil,

$$(6 \times 12)\left(100 + \frac{bd}{R}E_c\right) = 0{\cdot}537\left(60{,}000 - \frac{bd}{R}E_s\right).$$

$$\frac{bd}{R}E_c\ (72 + 0{\cdot}537\ m) = 25{,}000.$$

If $E_s = 28{,}000{,}000$ lb. per sq. in., $E_c = 4{,}500{,}000$ lb. per sq. in., and $m = 6{\cdot}22$,

$$\frac{bd}{R}E_c = \frac{25{,}000}{72 + 3{\cdot}34} = 332 \text{ lb. per sq. in.}$$

Thus the max. compression in the concrete when the tank is empty is 100 + 332

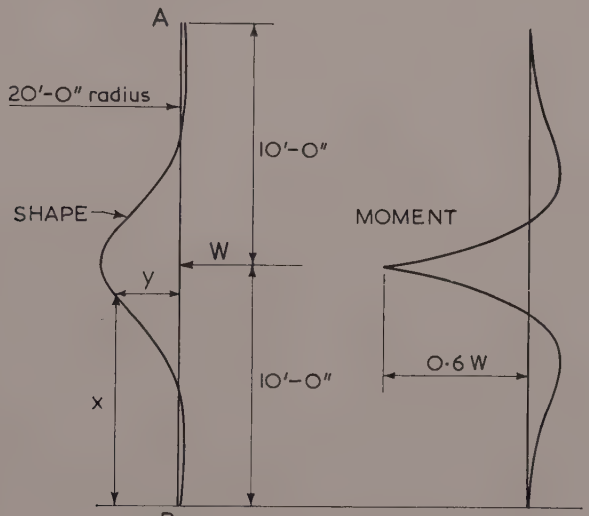


Fig. 28.2.

Fig. 28.3.

or 432 lb. per sq. in. at the foot of the wall and the tension in the steel at the foot of the wall when the tank is empty falls to

$$60{,}000 - (332 \times 6{\cdot}22) = 60{,}000 - 2{,}060 = 57{,}940 \text{ lb. per sq. in.,}$$

and this is the tension to which the hoop steel should be stressed.

All these are primary figures and take no account of losses due to friction, locking-off or non-elastic strain.

The actual distance bd is $\frac{332 \times (20 \times 12)}{4{,}500{,}000} = 0{\cdot}0177$ in. and $Bd = 0{\cdot}023$ in.

If all the hoop prestressing could be applied simultaneously and the tank filled instantaneously no further problem would arise. Generally, however, the hoops are stressed singly and this causes bending in the wall. If the straight line AB in *Fig.* 28.2 represents the unstressed wall in *Fig.* 28.1 and a single hoop half-way

up the wall is stressed with a tension equal to WR, this is equivalent to a load of W per foot run of the perimeter of the tank.

Taking the origin at B,

$$EI\frac{d^4y}{dx^4} = -C_2y, \text{ where } C_2 = \frac{E_cT}{R^2},$$

$$\text{giving } y = \left[e^{\frac{\alpha x}{\sqrt{2}}}\left(A \sin \frac{\alpha x}{\sqrt{2}} + B \cos \frac{\alpha x}{\sqrt{2}}\right) + e^{-\frac{\alpha x}{\sqrt{2}}}\left(C \sin \frac{\alpha x}{\sqrt{2}} + D \cos \frac{\alpha x}{\sqrt{2}}\right)\right]. \quad (16)$$

where $$\alpha = \sqrt[4]{\frac{E_cT}{R^2E_cI}} = \frac{1{\cdot}86}{\sqrt{TR}}.$$

If $T = 6$ in. $= 0{\cdot}5$ ft., and $R = 20$ ft.,

$$\alpha = \frac{1{\cdot}86}{\sqrt{0{\cdot}5 \times 20}} = 0{\cdot}588.$$

This is the simplest symmetrical case, giving the shape shown in *Fig.* 28.2 and the moment shown in *Fig.* 28.3, the maximum inward movement being 167 W/E_c

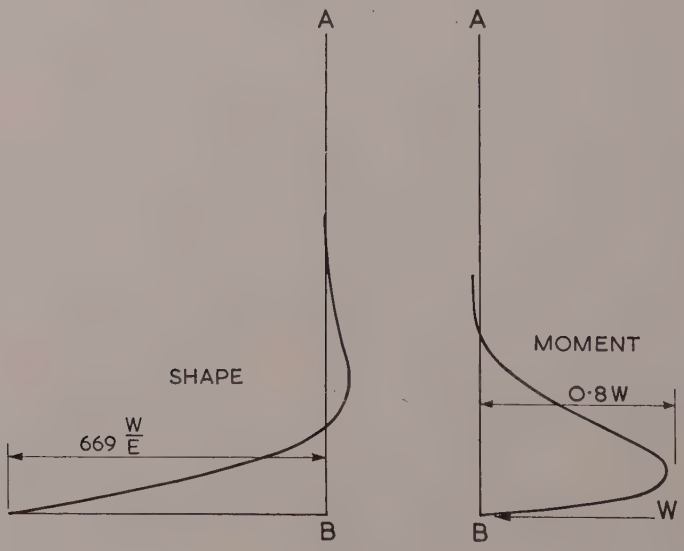


Fig. 28.4.

ft. and the maximum moment being $0{\cdot}6W$ lb.-ft. per foot run of perimeter, when W is expressed in pounds and E_c in pounds per sq. ft.

The next case to be considered is the application of a concentrated load of W lb. per foot run of perimeter to the base of the wall. The shape to which the wall deflects and the moments in the vertical 'staves' of the wall under such a load are shown in *Fig.* 28.4. In *Fig.* 28.5 the moment caused by a concentrated load W (that is, a hoop stressed to WR) placed 2 ft. from the base of the wall is shown. The calculation is more complicated than the previous cases but it can be solved by treating the bottom 2 ft. and the top 18 ft. as separate parts. Four coefficients are obtained in equation (16) for each part, say A, B, C and D for the top 18-ft. portion, and A', B', C' and D' for the bottom 2-ft. part. The fact that the ends of the wall are free gives $C = A$, $C' = A'$, $D = B - 2A$ and $D' = B' - 2A'$.

The values of y, $\frac{dy}{dx}$, $\frac{dy^2}{dx^2}$ and $\frac{dy^3}{dx^3}$ at the point of application of the load are then expressed firstly in terms of A and B and then in terms of A' and B'.

y (from below) $= y$ (from above). $\frac{dy}{dx}$ (from below) $= -\frac{dy}{dx}$ (from above).

$\frac{d^2y}{dx^2}$ (from below) $= \frac{d^2y}{dx^2}$ (from above).

$EI\frac{d^3y}{dx^3}$ (from below) $+ EI\frac{d^3y}{dx^3}$ (from above) $= -W$.

These four equations give A, B, A' and B'. The effect of a single concentrated load per foot run of perimeter gives the effect of stressing one single hoop, but it will

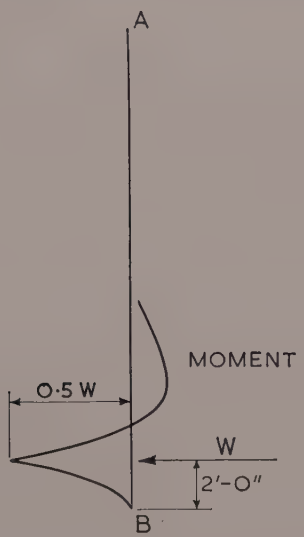


Fig. 28.5.

probably also be necessary to know the maximum moment caused by hoop-stressing any given height of wall. This can be found by extending the results already obtained and may be done by calculating the effects of applying a single hoop at depths of 2 ft., 4 ft., 6 ft., etc., and adding together the effects of any group of curves.

If hoops stressed to *WR* are applied to the tank in *Fig.* 28.1 at depths of 9 ft., 10 ft. and 11 ft., and the moment diagram for each drawn (as in *Fig.* 28.6) these can be added together to give a combined moment as shown in the figure. The value of this is about 1·0 *W*. If the applied hoop tension per foot of height is (985 × 20) lb., then $W = 985$ lb. and $M = 985$ lb.-ft. per foot run of perimeter.

Bending stress (ignoring steel) $= \frac{985 \times 12 \times 6}{12 \times 6^2} = \pm 164$ lb. per sq. in. (tension at water face).

If two hoops stressed to *WR* are applied at depths of 18 ft. and 20 ft., the separate moment diagrams are as already drawn in *Figs.* 28.4 and 28.5 and these may be combined as in *Fig.* 28.7, giving a possible maximum moment of 0·6 *W*. If the applied hoop tension per foot of height is (1,610 × 20) lb., then $W = 1{,}610$ lb.

and M = about 0·6 × 1,610 or about 1,000 lb.-ft. per foot run of perimeter (tension on outside face). If it is required to have a minimum compression of 100 lb. per sq. in. everywhere in the wall, a vertical compression of (164 + 100) lb. per sq. in. must be applied, giving a maximum compression of (264 + 164) = 428 lb. per

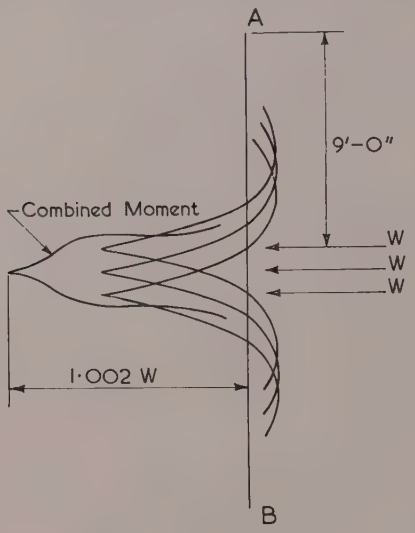


Fig 28.6.

sq. in. Whether more points and different combinations of loading should be examined depends on the size and importance of the tank but worked-out results are always useful for future reference.

If the system of hooping is such that no re-tensioning is possible, some thought should be given to the ratio between the tank diameter when each hoop is tensioned

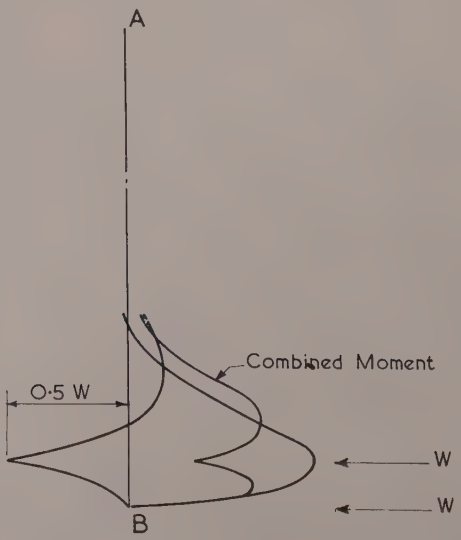


Fig. 28.7.

and the final diameter after all hoops are tensioned. If the hooping is tensioned progressively from the bottom the first hoop is applied when the wall is in position AB in *Fig.* 28.1 but as the hooping proceeds the point B moves to *d*. If *W* in *Fig.* 28.4 is taken as the moment of unit loading produced by a 1-ft. height of hooping at (1,610 × 20) lb. tension, the inward movement is

$$669\,\frac{W}{E} = \frac{669 \times 1{,}610 \times 12}{4{,}500{,}000 \times 144} = 0{\cdot}02 \text{ in.}$$

This is nearly as much as the distance B*d*, which is 0·023 in. If *W* in *Fig.* 28.2 is taken as 985 lb., the inward movement is 0·005 in. against a final inward movement of 0·0141 in.

The total loss of tension in a hoop when B moves to *d* in this example is 2,680 lb. per sq. in. or less than 5 per cent. of an initial stress of 60,000 lb. per sq. in. This is probably much less than the errors that are likely to be made in assessing the value for *m* and estimating the amount of inelastic strain. In this example the inward movement of the wall has been calculated for a concentrated load due to single prestressing hoops at depths of 10 ft. and 20 ft. In an important case this can be extended to other points so that the shape of the wall at all stages of construction can be calculated. The work involved may be reduced by using Maxwell's Theorem.

Uniform Wall Fully Restrained at Base

As a simple example of the problems involved consider the tank in *Fig.* 28.1 and imagine the bottom end of the wall fully-restrained both in position and direction as shown in *Fig.* 28.8. In addition, assume that the same hoop steel supplied for the unrestrained tank wall in *Fig.* 28.1 is also supplied for the restrained wall in *Fig.* 28.8. This may not be the most economical arrangement but will give a quick first approximation to the maximum stresses and is in fact recommended in

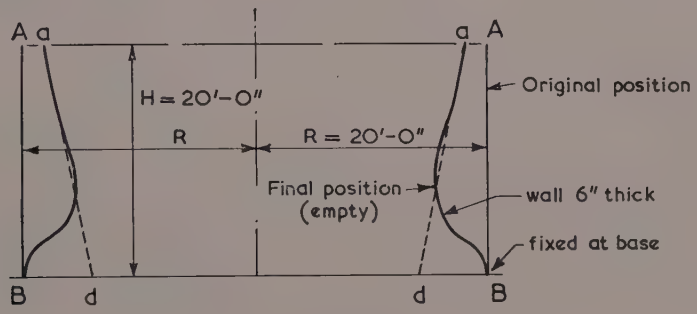


Fig. 28.8.

CP.2007:1960. The loading on the wall produced by the hoops when the tank is empty is sufficient to balance the water pressure and produce a compression of 100 lb. per sq. in. in the concrete, which last item is equivalent to a uniform external pressure of 360 lb. per sq. ft.

$$H/\sqrt{TR} = 20/\sqrt{(0{\cdot}5 \times 20)} = 6{\cdot}33.$$

The restraint moment and restraint shear for a tank of these proportions may be obtained at once from tabulated values. The restraint moment and shear due to a uniform pressure *p* may readily be calculated.

Total restraint moment

$$= 0{\cdot}255wHTR + 0{\cdot}29pTR$$

$$= 0{\cdot}255 \times 62{\cdot}5 \times 20 \times 0{\cdot}5 \times 20 = 3{,}190 \text{ lb.-ft.}$$
$$+ 0{\cdot}29 \times 360 \times 0{\cdot}5 \times 20 = 1{,}045 \text{ lb.-ft.}$$
$$4{,}235 \text{ lb.-ft. per foot run.}$$

The moment causes tension on the outside face of the wall.

Total restraint shear $= 0{\cdot}112wH^2 + 0{\cdot}128pH$

$$= 0{\cdot}112 \times 62{\cdot}5 \times 20^2 = 2{,}800 \text{ lb.}$$
$$+ 0{\cdot}128 \times 260 \times 20 = 920 \text{ lb.}$$
$$3{,}720 \text{ lb. per foot run.}$$

Bending stress at base of wall (tank empty)

$$= \pm \frac{4{,}235 \times 12 \times 6}{12 \times 6^2} = \pm 707 \text{ lb. per sq. in. (tension on outside face).}$$

The maximum reversed moment that occurs further up the wall is very nearly

$$-0{\cdot}06 \times 62{\cdot}5 \times 20 \times 0{\cdot}5 \times 20 = 750 \text{ lb.-ft.}$$
$$-0{\cdot}06 \times 360 \times 0{\cdot}5 \times 20 = 216 \text{ lb.-ft.}$$
$$= 966 \text{ lb.-ft. per foot run.}$$

Bending stress due to reversed moment

$$= \pm \frac{966 \times 12 \times 6}{12 \times 6^2} = \pm 161 \text{ lb. per sq. in. (tension on water face).}$$

In order to maintain a minimum compression of 100 lb. per sq. in. in a vertical direction a compression of 807 lb. per sq. in. must be applied down the outside face and 261 lb. per sq. in. down the water face. A compressive force of 38,400 lb. per foot run of wall placed 2·49 in. from the outside face will produce this. This will give values of maximum compression of 968 lb. per sq. in. on the water face at the base of the wall and on the outside face about 3 ft. 6 in. above the floor.

If the base of the wall is restrained in position ring compression cannot be produced at the extreme bottom unless the diameter of the floor of the tank is reduced by prestressing. This would reduce the restraint moment and shear in the same way as the stretching of the floor of a tank of normal reinforced concrete.

If it is desired to examine the deflections and moments in the wall during the process of prestressing the moments and deflections caused by a single hoop placed at any height can be calculated by applying equation (16) firstly to the part above the hoop and then to the part below. When dealing with the part below the hoop the end conditions are $y = 0$ when $x = 0$ and $\frac{dy}{dx} = 0$ when $x = 0$, giving $D = -B$ and $C = -A - 2B$ for this part of the wall.

Uniform Wall Hinged at Base

Reference is made in CP.2007:1960 to what is vaguely described as a ' pinned foot '. The writer has no clear idea of what this means. The case of a wall completely free to slide at the bottom has already been discussed as has that of a wall fixed in position and direction. Now the case where the foot of the wall is fixed in position but not in direction, as shown in *Fig.* 28.9, will be considered.

The effect of the hooping sufficient to contain the water pressure may be found by the method used for the same problem with tanks of normal reinforced concrete. The moments in the lower part of the wall of this tank, which has the ratio $H/\sqrt{TR} = 6{\cdot}33$, are shown in *Figs.* 28.10 and 28.11. The effect of hooping to

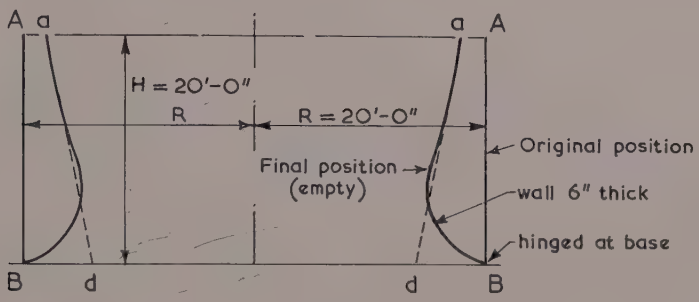


Fig. 28.9.

contain the water pressure is shown in *Fig.* 28.10 and in *Fig.* 28.11 the effect is shown of the provision of additional hooping to exert an external pressure p equal to 360 lb. per sq. ft. to produce a residual hoop compression of 100 lb. per sq. in. when the tank is full. The values in *Fig.* 28.11 are arrived at as follows:

$$R = 20 \text{ ft.} \quad H = 20 \text{ ft.} \quad T = 0{\cdot}5 \text{ ft.}$$

$$\frac{d^4y}{dx^4} = \frac{p}{EI} - \frac{yT}{R^2I}.$$

$$y = \frac{p}{\alpha^4 EI} + \left[e^{\frac{\alpha x}{\sqrt{2}}}\left(A \sin \frac{\alpha x}{\sqrt{2}} + B \cos \frac{\alpha x}{\sqrt{2}}\right) + e^{-\frac{\alpha x}{\sqrt{2}}}\left(C \sin \frac{\alpha x}{\sqrt{2}} + D \cos \frac{\alpha x}{\sqrt{2}}\right)\right],$$

where $\alpha = 1{\cdot}86/\sqrt{TR} = 1{\cdot}86/\sqrt{0{\cdot}5 \times 20} = 0{\cdot}588$.

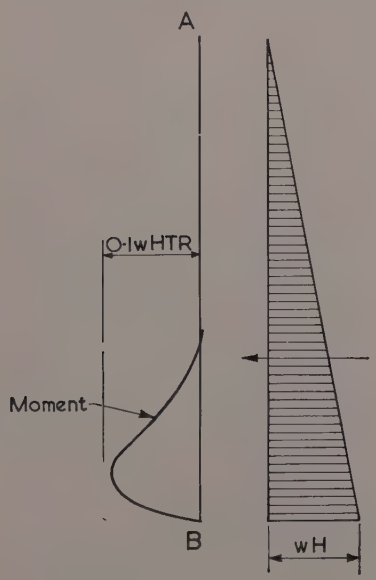


Fig. 28.10.

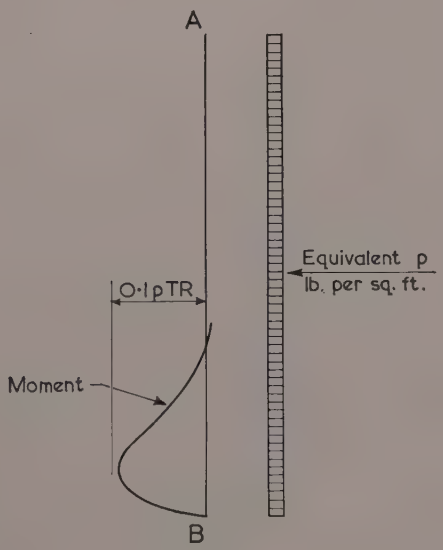


Fig. 28.11.

Taking the origin for x at the top of the wall,

$\frac{d^2y}{dx^2} = 0$ when $x = 0$ and $\frac{d^3y}{dx^3} = 0$ when $x = 0$,

giving $C = A$ and $D = B - 2A$.

also $y = 0$ when $x = H = 20$ ft. . . . . (f)

$\frac{d^2y}{dx^2} = 0$ when $x = H = 20$ ft. . . . . (g)

When $x = 20$ ft., $\frac{\alpha x}{\sqrt{2}} = \frac{0{\cdot}588 \times 20}{1{\cdot}414} = 8{\cdot}32$ radians.

$e^{8{\cdot}32} = 4{,}105{\cdot}2$. $e^{-8{\cdot}32} = 0{\cdot}000244$. $\sin 8{\cdot}32 = +0{\cdot}8935$. $\cos 8{\cdot}32 = -0{\cdot}4491$.
$e^{8{\cdot}32} \sin 8{\cdot}32 = +3{,}666$. $e^{-8{\cdot}32} \sin 8{\cdot}32 = +0{\cdot}000218$.
$e^{8{\cdot}32} \cos 8{\cdot}32 = -1{,}844$. $e^{-8{\cdot}32} \cos 8{\cdot}32 = -0{\cdot}000110$.
The value of $e^{-8{\cdot}32}$ is so small that the coefficients for C and D disappear from slide-rule calculations in equations (f) and (g).

Equation (f) becomes $0 = \frac{p}{\alpha^4 EI} + [3{,}666A - 1{,}844B]$ . . . . (f)

and equation (g) becomes $0 = -1{,}844A - 3{,}666B$ . . . . . (g)

giving $1{,}000\ A = -0{\cdot}2175\ \frac{p}{\alpha^4 EI}$ and $1{,}000\ B = +0{\cdot}1095\ \frac{p}{\alpha^4 EI}$.

By assuming values of x of 2 ft., 4 ft., 6 ft., etc., any values of y, $\frac{d^2y}{dx^2}$ and $\frac{d^3y}{dx^3}$ that are required to obtain the shape, moment or shearing force at any point may be obtained.

For the tank in *Fig.* 28.9 the maximum moment in the wall when the tank is empty (from *Figs.* 28.10 and 28.11) is $(0{\cdot}0927wHTR) + (0{\cdot}0927pTR)$

$= 0{\cdot}0927 \times 62{\cdot}5 \times 20 \times 0{\cdot}5 \times 20 = 1{,}160$
$+ 0{\cdot}0927 \times 360 \times 0{\cdot}5 \times 20 = 334$
$1{,}494$ lb.-ft. per foot run,

(tension on water face).

In Paragraph 318 in CP.2007:1960, it is stated that ' when the foot of the wall is free to slide a longitudinal moment should be assumed on the basis of a restraint equal to one-half of that provided by a pinned foot '.

This apparently means that when the foot of the wall is provided with a sliding joint of asphalt together with a metal or plastic water-bar, the vertical ' staves ' of the wall should be designed for a moment equal to one-half the moment in a wall where the foot is restrained in position but not in direction; in this particular tank, one-half of 1,494 lb.-ft. per foot run of perimeter.

In a large tank it may be necessary to calculate the deflection of and the moments in the wall when only part of the hooping is prestressed. The method of calculating the effect of a single hoop (given earlier in connection with a wall free to slide at the base) can again be followed.

Stability of a Thin Circular Wall

A thin circular ring subjected to an external pressure p becomes unstable when $p = 3EI/R^3$. If $E = 4{,}500{,}000 \times 144$ lb. per sq. ft., $I = \frac{1}{12} \times 1 \times T^3$ ft.^{4} per foot

height of wall, p = pressure per square foot in pounds, and R = radius in feet,

$$p = \frac{3 \times 4{,}500{,}000 \times 144}{R^3} \times \frac{T^3}{12} \text{ or } T = R \times \sqrt[3]{\frac{p}{162{,}000{,}000}}.$$

If $R = 20$ ft. and $p = 1{,}250$ lb. per sq. ft.,

$$T = 20 \times \sqrt[3]{\frac{1{,}250}{162{,}000{,}000}} = 0{\cdot}396 \text{ ft.} = 4{\cdot}75 \text{ in.}$$

If $R = 20$ ft. and $p = 625$ lb. per sq. ft. then $T = 3{\cdot}75$ in.

Thus if a circular concrete wall 20 ft. radius and 20 ft. high sitting on a frictionless sliding joint and varying in thickness from 4·75 in. at the bottom, to 3·75 in. at halfway up, to nothing at the extreme top is constructed, and is subjected to external water pressure it will be just on the point of collapsing inwards. This assumes that the concrete is perfectly uniform and the wall perfectly circular on plan. Allowing a design factor of 2 to cover variations in construction and assuming a factor of safety of 2, the wall thickness would need to be increased in the ratio of $\sqrt[3]{2 \times 2}$; that is, an increase of 59 per cent.

Although the addition of prestressed hoops has the same primary effect as applying external water pressure, the secondary effects are quite different. The intensity of water pressure is unaffected by the shape of the tank, whereas the intensity of pressure exerted by a tensioned hoop varies inversely as the radius of curvature. If the wall is slightly oval, water pressure will tend to make it more so, while a tensioned hoop will tend to pull it circular but will, of course, produce bending moments in the wall in so doing. If the wall is 6 in. in thickness and if it is so far out of circular that the position of the centre of compression at any point is 1 in. out from the centre of the wall the intensity of ring compression will be doubled. The same kind of effect is produced if the compaction of the concrete varies from the inside to the outside face of the wall. From this it appears that the shuttering must be very accurately made, erected and maintained in position and it is unwise to use a mix of concrete which is difficult to consolidate.

Construction and Useful Life of Prestressed Tanks

At the present time (1967) most, if not all, prestressed work is built on some proprietary system or by the use of proprietary apparatus. The stress losses due to friction, locking-off and inelastic strain depend on the system and the type of reinforcement used.

The earliest form of prestressed circular tank had mild steel bars with upset threaded ends joined by turnbuckles placed round the outside of the wall, and was a direct extension of the technique of making large wooden vats. This technique is still largely employed, the hoops (now of high-tensile steel) being protected by a layer of gunite. If this gunite is applied when the tank is empty the subsequent stretching of the wall will place it in tension. This suggests the use of lightweight aggregate with a low elastic modulus, so that the gunite can stretch without cracking, with a good tack coat between the wall and the gunite.

A serious question must now be considered—the deterioration of prestressed concrete tanks. In this respect a reference back to the early structures of normal reinforced concrete is illuminating. The concrete cover shown on the drawings for watertight or marine work was sometimes as little as $\frac{1}{2}$ in. and, since effective

spacers were not commonly used, the actual cover was often less—sometimes only a coat of cement wash. In these early days the use of overwet or overlean concrete mixes sometimes produced porous concrete that made matters worse. Enthusiastic pioneers placed the effective life of these early structures at more than a hundred years, and when many of them showed ominous rust marks, and ' blows ' before they were twenty years old the enthusiasts continued to argue ferociously against the facts. Since human nature does not change, history repeats itself. Is not, in fact, our present attitude to prestressed work based more on enthusiastic optimism than on a cold appraisal of the facts?

It is difficult to avoid occasional air locks in pipes carrying water and more difficult, if not impossible, to avoid air pockets in ducts containing cables and grounted with cement or some bituminous liquid. If a harsh dry mix that gives very high test-cube strengths is used, it is very difficult to eliminate honeycoming along the underside of bars or ducts and this honeycoming could form internal channels to lead the water to the steel. It is much easier to consolidate concrete around vertical or near-vertical bars or ducts than around horizontal ones. Whether it is better to place the bars or cables in ducts where they could deteriorate to the point of failure without warning or to place them externally where serious rusting would show and where they could be exposed and inspected is a question that the designer must decide after careful consideration of site conditions.

In April 1961 a 10-years' old prestressed tank in New York collapsed without warning. Whether this is an isolated case or the precursor of more failures time alone will show. The reader is advised to study the reports of the incident.

Prestressed Tanks at Very Low Temperatures

The suitability of prestressed tanks for storing very cold liquids, such as liquid methane, depends on providing a concrete that is impervious to the type of liquid-to-gas leakage that occurs through concrete walls and on using a type of steel that can withstand repeated cycles of stress at very low temperatures.

If the prestressed tank is merely an outer container to hold a fully insulated impervious inner container, these problems do not arise.

CHAPTER TWENTY-NINE

REPAIRS

Repairs to reinforced concrete are always expensive and often unsatisfactory. It should be borne in mind that money spent on endeavouring to reduce leakage to a very small amount by the application of rendering or other protective coating may exceed the capitalized value of the escaping water.

When a reinforced concrete structure deteriorates the design and specification should be examined as well as the structure itself. Undetected mathematical errors in design are rare. If the design is at fault it is usually because of a fundamental error such as over-estimation of the safe bearing capacity of the ground or of the minimum lateral pressure exerted by the ground. Mistakes in detailing (for example, overcrowding of bars and high local bond stresses) are more frequent. Mistakes in specification are also much too frequent; these include failure to prepare a proper concreting programme, failure to specify positive methods for keeping reinforcement in position, failure to insist on proper curing and protection of the concrete, and the use of unsuitable water–cement ratios or gradings of aggregates. The bill of quantities may also be at fault. Unusual items in the specification should also appear in the bill of quantities as separate units to be priced.

Shortcomings in construction are usually obvious; they may include poor workmanship at joints, inadequate cover, and poor consolidation of the concrete. Local cracks or leaks can also be caused by accidents such as the striking of upstanding splice-bars in newly-set concrete by mechanical plant. It is therefore necessary to decide whether a defect is the result of an isolated accident (such as a local patch of bad workmanship) or whether it is due to a more fundamental error and is likely to occur at all similar points. For example, the reinforcement in one counterfort may be found to be rusting and spalling off the concrete; an examination of the details may show the reinforcement at this position to be so crowded that the concrete could not properly be compacted, the consequent honeycoming allowing the water to penetrate to the reinforcement. If all the counterforts were built in the same way it would be highly probable that similar trouble would develop in all of them, and widespread remedial measures would be necessary instead of an isolated repair.

Most tanks and reservoirs are tested by means of water pressure, and in many cases no back-filling is allowed until the test has been satisfactorily concluded. The allowable leakage from a reservoir during a test is often required to be not more than $\frac{1}{8}$ in. in a definite period, usually between the seventh and fourteenth day after first filling with water.

Seepage will often stop quickly if the water contains finely-divided matter in suspension, or as the result of chemical action between the concrete and dissolved carbon dioxide. In some cases, also, sealing is due to hydration of unset cement.

If visible leaks develop in the walls they can be marked and recorded, but if no such leaks are seen and yet the water-level drops more than the allowable amount then the floor is probably not watertight, and the points or planes of leakage must be guessed.

Cracks in Walls of Reservoirs above Ground

The most common cause of leakage is the opening of construction joints, generally the horizontal joints and in particular the lowest horizontal joint. Usually the reservoir can be emptied and the repairs effected on the inside (pressure) face. Leaky joints should be cut out to a depth of 1 in. or $1\frac{1}{4}$ in.; this is best done by means of a power-driven carborundum saw which can be used to produce a square or even slightly undercut groove (*Fig.* 29.1). The saw blades are rapidly worn away, but the cutting of a V-shaped groove by hand is expensive and rarely successful. The groove is coated with mastic or bitumen and then filled with cement mortar or synthetic resin.

Another and simpler method is to clean and dry the surface of the wall for a width of about 6 in. on each side of the crack and apply a thin layer of bitumen about 9 in. wide. On this is laid a continuous strip of aluminium foil about 6 in.

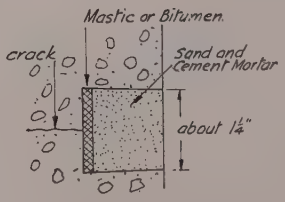


Fig. 29.1.

wide, which in turn is covered with another layer of bitumen. This method can also be used as an extra safeguard to cover the repair shown in *Fig.* 29.1.

If the emptying of the reservoir is not permitted, minor repairs may be attempted on the outside face against the pressure, but the task is more difficult. If the weeping is only slight the method shown in *Fig.* 29.1 may be tried, but quick-setting cement must be used. A method which is permissible for small repairs of this nature is to mix small quantities of Portland cement and high-alumina cement, wet the resulting mixture quickly, apply it immediately, and hold a board against the surface for a few minutes until the mixture has set hard. There are now on the market synthetic polymers which, when mixed together, harden almost immediately. These may be used when there is a sizable opening into which they can be forced.

If these measures fail then the reservoir must be emptied.

Cracks in Walls of Underground Tanks

Leakage of water from the subsoil into an empty or partly-empty tank is most easily dealt with by excavating outside the walls and reducing the level of the ground water by pumping; failing this, grouting-tubes may be driven outside the tank and cement grout (or other consolidating fluids) pumped into the ground to form an outside repair, care being taken not to develop sufficient pressure to break the tank. The insides of the cracks may then be made good as shown in *Fig.* 29.1.

If grouting-tubes cannot be driven outside the tank, holes may be drilled through the walls along the lines of the cracks and the grouting-tubes passed through them. This work is now normally done by specialist firms.

If the walls are not considered to be sufficiently strong to withstand grouting,

holes may be drilled through them near the cracks. Pipes of about 2 in. diameter are set in these holes, caulked with tarred hemp and lead-wool, and firmly fixed in quick-setting mortar. This localizes the flow, relieves the pressure on the wall, and makes it possible to repair the cracks from the inside. The pipes should then be sealed with screw-caps or plugs. Alternatively the ground-water level could be lowered by sinking well-points outside the walls.

Repairs to Floors

On a reasonably dry and firm site where only a thin floor is provided it may be known that water is leaking through the floor, although the positions of the leaks may not be apparent. If an examination of the design and specification indicates that porosity or honeycombing is very unlikely, then the construction joints may be treated by one of the methods described for the walls. If this does not reduce leakage to a negligible amount the whole floor may have to be lined. If the floor can be thoroughly dried it may be cleaned and lined with asphalt. Otherwise the surface could be hacked and covered with a granolithic rendering 2 in. thick reinforced with steel mesh, or given a coating of gunite.

At a wet site, where there is permanent water-pressure under the floor, water may enter from below when the reservoir is empty. With the type of thick floor needed in such conditions this leakage will occur only through the joints. These may be sealed by drilling holes and passing grouting-tubes to the underside. If grouting is not possible, pressure-relieving pipes may be fixed as previously described and repairs attempted from the top using the method shown in *Fig.* 29.1. If local repairs fail it may be necessary to place another thick floor over the first, which greatly increases the cost and may seriously reduce the capacity.

General Repairs to Walls

In modern construction, using 1 : 1½ : 3 concrete and rounded aggregate of reasonably good quality, the only possible source of porosity is honeycombing due to the use of concrete with a low water content or to the overcrowding of the reinforcement. If an examination of the design and specification indicates that such conditions are likely throughout the walls, the local repair of the first damp patches would probably be of little use and a complete waterproof covering would be required. In an old reservoir, in which the reinforcement is too close to the surface and has commenced to rust, causing spalling, the damage is probably much more extensive than is at first apparent and extensive cutting-away and renewal is usually necessary. If the cutting-away is so severe as seriously to reduce the strength, the use of gunite should be considered. A minimum thickness of 1½ in., reinforced with steel mesh, should be specified, the mesh being plugged into the old work.

In new work where no deterioration has developed but where porosity is apparent or suspected, a layer of aerated mortar ½ in. in thickness may be sprayed on. This is usually preceded by a tack coat of an organic polymer such as polyvinyl acetate. Synthetic resins are also being developed for protective coatings. Stabilized bitumen, sometimes reinforced with a scrim of glass fibre, has been used as a waterproof covering for reinforced concrete buildings, but its continued adhesion to concrete in submerged work is doubtful.

A reservoir of 12,000,000 gallons capacity in County Durham which was severely

damaged as a result of mining subsidence has been lined throughout with high-grade rubber $\frac{1}{4}$ in. thick; so far this has been successful, but it has not yet been subjected to severe movement of the ground.

Repairs to Roof

Deep narrow beams overcrowded with reinforcement are very soon attacked in a wet atmosphere and their repair usually entails extensive cutting away and building up with gunite. Cracks in the roof slab, although not subjected to water pressure, may open and close with changes in the temperature of the air. The filling of such cracks when they are at their widest may cause other cracks to develop, and possibly the best treatment is to cover them with strips of aluminium foil set in bitumen.

APPENDIX ONE

SOME NOTES ON BRITISH STANDARD CODE OF PRACTICE CP. 2007:1960

THE DESIGN AND CONSTRUCTION OF REINFORCED AND PRESTRESSED CONCRETE STRUCTURES FOR THE STORAGE OF WATER AND OTHER AQUEOUS LIQUIDS

To understand the position better regarding the recommendations contained in CP.2007: 1960 the reader is advised to look back on the unhappy history of official regulations in this country. By 1913 many large reinforced concrete structures had already been built. At that time the only document that could be said to have any official status was the Second Report published by the Royal Institute of British Architects in 1911. It should be particularly noted that this document was not published by the reinforced concrete industry nor even by the Concrete Institute or the Institution of Civil Engineers, but was sponsored by a body essentially architectural. This was, of course, a most unhappy start and all branches of the industry were to blame for failing actively to co-operate in producing something sound and practical. In 1915 the London County Council published mandatory Regulations. These were greeted with dismay and derision by the firms who specialized in

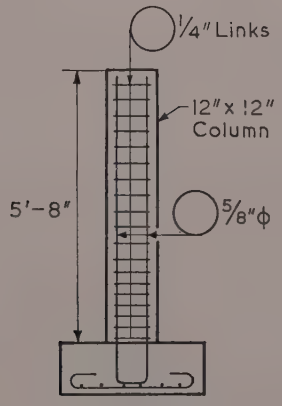
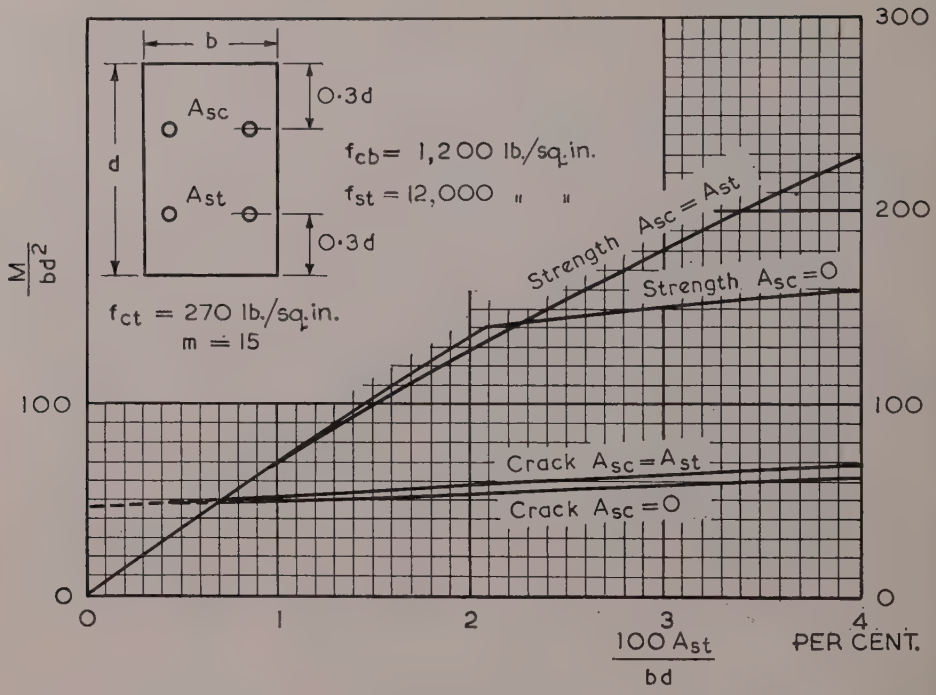


Fig. A1.1.

design and construction. The reader will be surprised to learn that (according to these Regulations) the column shown in *Fig.* A1.1 will collapse under its own weight.

Modern codes of practice really stem from the Report of 1933 issued by the Department of Scientific and Industrial Research. Here again is a document not published by the specialist firms who introduced and developed reinforced concrete in this country and not published by the Institution of Civil Engineers but sponsored by a Department engaged largely on small-scale laboratory research. Again many specialist firms were to blame. When asked to complete and return questionnaires, they did not see why they should divulge information which they had acquired through years of hard experience. Those who were most niggardly in supplying information were the first to complain that the new codes of practice were unpractical. And so the unfortunate discord continues. Engineers who have learned their design the hard way with competitive design-and-contract firms consider the


b
d
Asc
Ast
0·3d
0·3d
fcb= 1,200 lb./sq.in.
fst = 12,000 " "
fct = 270 lb./sq.in.
m = 15
M/bd²
Strength Asc = Ast
Strength Asc = 0
Crack Asc = Ast
Crack Asc = 0
300
200
100
0
100
0
1
2
3
4
100 Ast/bd
PER CENT.

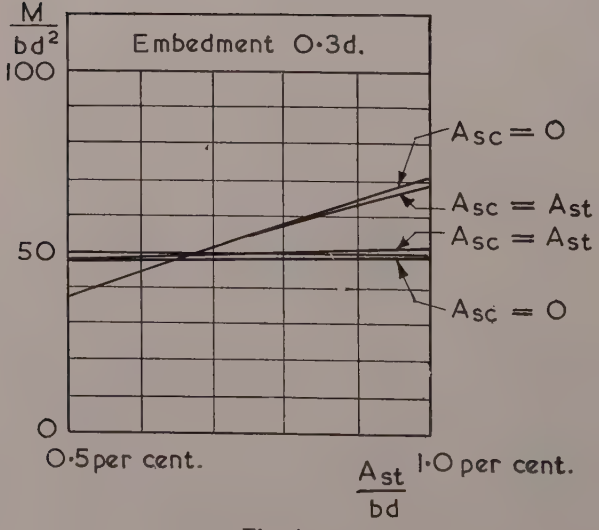

M/bd²
100
50
0
Embedment 0·3d.
Asc = 0
Asc = Ast
Asc = Ast
Asc = 0
0·5 per cent.
Ast/bd
1·0 per cent.

Fig. A1.2.

restrictions on the permissible tensile stresses in bending in CP.2007:1960 unnecessarily severe. But to do nothing beforehand and then to complain bitterly afterwards is little help towards a satisfactory solution.

It is amusing to note that a foundation slab under a building standing on waterlogged ballast may, according to CP.114: 1957, be reinforced with steel stressed to 30,000 or even 33,000 lb. per sq. in. Under similar circumstances CP.2007:1960 would limit the stress to 12,000 lb. per sq. in.—a mere discrepancy of 175 per cent.

Code Recommendations

Concrete

For all members thinner than 18 in., the Code recommends a concrete comprising 112 lb. of cement to 2 cu. ft. of sand to 4 cu. ft. of coarse aggregate, that is a nominal 1 : 1·6 : 3·2 mix. For thicker members, the mix recommended is 112 lb. to $2\frac{1}{2}$ cu. ft. to 5 cu. ft. or a nominal 1 : 2 : 4 mix.

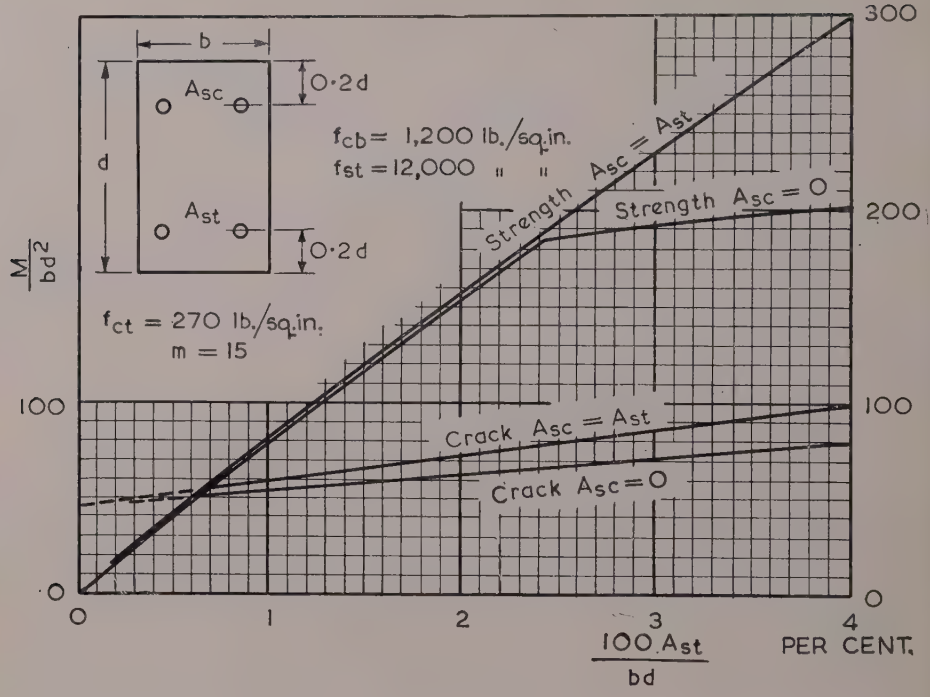
Members in Tension Only

For members subjected to direct tension (such as the wall of a circular tank without bottom restraint), the Code recommends that (1) there should be sufficient reinforcement to resist all the tension at a stress of not more than 12,000 lb. per sq. in.; and (2) assuming that the concrete is uncracked, the modular ratio is 15, and concrete and reinforcement act together, the calculated tensile stress in the concrete should not exceed 190 lb. per sq. in. for a 1 : 1·6 : 3·2 mix or 175 for a 1 : 2 : 4 mix. Presumably a tensile stress of 200 lb. per sq. in. would be allowed in the more common 1 : $1\frac{1}{2}$: 3 mix.

From condition (1), if T lb. is the total tension in a strip of wall 1 ft. wide, the area of reinforcement A_{st} required is $\frac{T}{12{,}000}$ sq. in. per foot. From condition (2), the thickness d (in.) of the wall should be such that $12dp_{ct} + (15 - 1)A_{st}p_{ct}$ should be not less than T. If $A_{st} = \frac{T}{12{,}000}$ and p_{ct} = 175 lb. (for 1 : 2 : 4 mix) 190 lb. (for 1 : 1·6 : 3·2 mix), or 200 lb. (for 1 : $1\frac{1}{2}$: 3 mix) per sq. in., $d = \frac{T}{2{,}500}$, $\frac{T}{2{,}930}$ or $\frac{T}{3{,}130}$ respectively, in which T is in pounds and d is in inches. Many existing tanks are designed for tensile stresses of from 13,500 lb. to 16,000 lb. per sq. in. in the reinforcement and a wall thickness of $\frac{T}{5{,}220}$, which is much less than the thickness required if the limiting tensile stress in the concrete, as recommended in the Code, is adopted.

Members Subject to Bending Only: Tension at Face in Contact with Liquid

The Code recommends the two following requirements: (1) For 1 : 1·6 : 3·2 concrete, a compressive stress of 1,200 lb. per sq. in. in the concrete and a tensile stress of 12,000 lb. per sq. in. in the reinforcement should not be exceeded. The tensile strength of the concrete is ignored in this calculation. The modular ratio is assumed to be 15. The curves marked 'Strength' in *Figs.* A1.2 to A1.4 indicate the bending resistance of rectangular sections reinforced in tension only or in tension and compression for three positions of the reinforcement. *Fig.* A1.2 usually applies to slabs about 5 in. thick, *Fig.* A1.3 to slabs about 9 in. thick, and *Fig.* A1.4 to slabs about 20 in. thick. (2) In addition to the foregoing, assuming that the concrete is uncracked and acts in unison with the steel and that m = 15, the tensile stress in the concrete should not exceed the safe tensile stress p_{ct} in bending. This provision is intended to ensure the absence of cracks. The bending strength of rectangular sections with p_{ct} equal to 270 lb. per sq. in. are shown by the curves marked 'Crack' in *Figs.* A1.2


b
d
Asc
Ast
0·2d
0·2d
fcb = 1,200 lb./sq.in.
fst = 12,000 " "
fct = 270 lb./sq.in.
m = 15
M/bd²
Strength Asc = Ast
Strength Asc = 0
Crack Asc = Ast
Crack Asc = 0
300
200
100
0
100
0
1
2
3
4
100 Ast/bd
PER CENT.

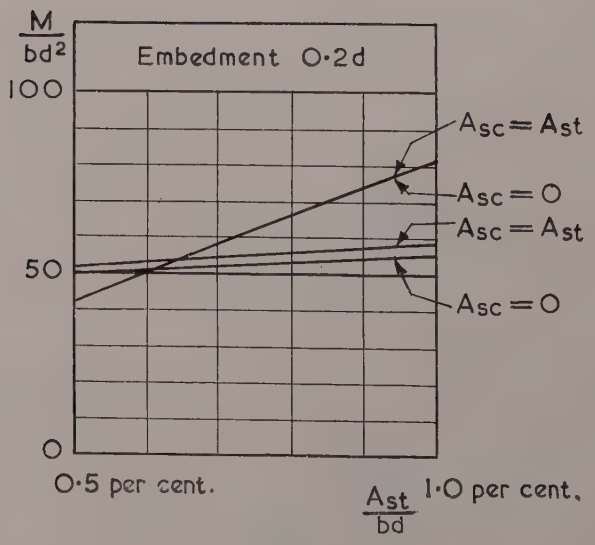

M/bd²
Embedment 0·2d
100
50
0
Asc = Ast
Asc = 0
Asc = Ast
Asc = 0
0·5 per cent.
Ast/bd
1·0 per cent.

Fig. A1.3.

to A1.4. Except when the percentage of reinforcement is very small, these curves are much below those marked 'Strength'. If any other safe tensile stress is specified, the moment of resistance factor with any given percentage of reinforcement is obtained by multiplying the factor on the diagrams by $\frac{p_{ct}}{270}$. The Code recommends values of p_{ct} of 270 lb. per sq. in. for 1 : 1·6 : 3·2 concrete and 245 lb. per sq. in. for 1 : 2 : 4 concrete. Presumably 300 lb. per sq. in. would be allowed for 1 : 1½ : 3 concrete.

For a rectangular section with reinforcement in tension only, with working stresses of 1,200 lb. per sq. in. in the concrete and 12,000 lb. per sq. in. in the reinforcement (with $m = 15$), $M/bd_1^2 = 288$, and the reinforcement required is 3 per cent. of the area bd_1. Because the second recommendation is based on the overall depth d, whereas the first is based on the effective depth d_1, the ratio d_1/d is important. The bending strengths in *Figs.* A1.2 to A1.4 are therefore shown in terms of bd^2 and the percentage of reinforcement in terms of bd. In *Fig.* A1.4, for example, $d_1 = 0{\cdot}9d$ and if the area of tension reinforcement is 3 per cent. of bd_1 (that is 2·7 per cent. of bd) and $A_{st} = 0$, then $M/bd^2 = 233$, corresponding to $M/bd_1^2 = 288$. For the same section, the value of M/bd^2 given by the curve marked 'Crack' is about 92, which is only about 40 per cent. of that given by the curve marked 'Strength'. It follows, therefore, that design for 'no cracking' determines the thickness of the slab, and design for 'bending strength' determines the amount of reinforcement.

In the upper parts of *Fig.* A1.2 to A1.4, the whole range of percentages of reinforcement up to 4 per cent. is shown, but only those values between 0·5 and 1·0 per cent. are generally of practical interest. This range is shown to a larger scale in the lower parts of *Figs.* A1.2 to A1.4.

Division Wall.—A division wall in a tank is subjected to a resultant pressure on either face if the head of water on one side differs from that on the other, or if one part of the tank is empty when the other is full or partially full. Reinforcement must therefore be provided on both faces.

EXAMPLE.—A division wall in a tank is to be designed as a cantilevered slab to resist a head of 15 ft. of water acting on either side.

The bending moment at the base is

$$M = \tfrac{1}{6} \times 62{\cdot}5 \times 15^3 = 35{,}000 \text{ ft.-lb.} = 422{,}000 \text{ in.-lb. per foot.}$$

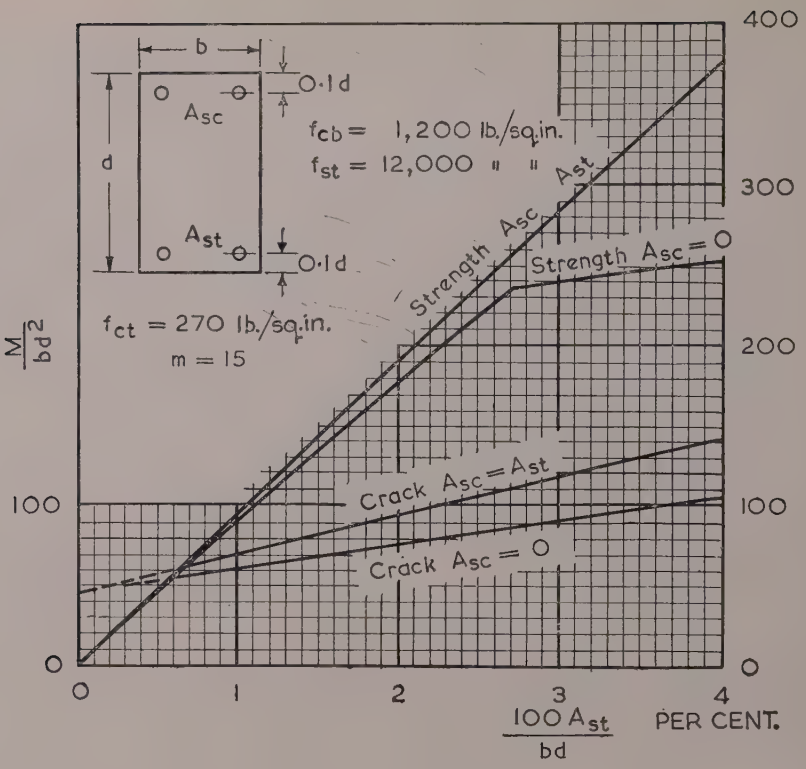
Assume d_1/d to be 0·9. The curves marked 'Strength $A_{sc} = A_{st}$' and 'Crack $A_{sc} = A_{st}$' in *Fig.* A1.4 intersect at $M/bd^2 = 60$, and the amount of reinforcement required at both faces of the wall is 0·62 per cent. The least permissible overall depth to prevent cracking is given by $d = \sqrt{\frac{422{,}000}{60 \times 12}} = 24{\cdot}2$ in., say 24½ in. overall.

$A_{sc} = A_{st} = 0{\cdot}62$ per cent. $= \frac{0{\cdot}62}{100} \times 12 \times 24{\cdot}2 = 1{\cdot}8$ sq. in. per foot. Provide 1-in. diam. bars at 5-in. centres (1·88 sq. in.) near each face. If the cover of concrete is 1½ in., $d_1 = 22{\cdot}5$ in. $= 0{\cdot}92d$; if the cover is 2 in., $d_1 = 22$ in. $= 0{\cdot}9d$. Both values are sufficiently close to the assumed value of $0{\cdot}9d$.

The observance of the 'no-cracking' rule leads to a much thicker wall than is required for strength alone, but this does not mean that such a design is uneconomical. In the example given, with stresses of 1,200 lb. per sq. in. in the concrete and 12,000 lb. per sq. in. in the reinforcement, sufficient strength could be obtained with a wall 13 in. thick reinforced with 1-in. diam. bars at 2⅜-in. centres near both faces. Assuming that the wall tapers uniformly to a thickness of 7 in. at the top, the average thickness would be reduced from 15¾ in. to 10 in. This would mean a saving of 36 per cent. of the concrete (0·27 cu. yd. per foot of wall) but an increase of about 100 per cent. in the main reinforcement (say 1·2 cwt. per foot of wall); thus the former design is likely to be the more economical.

Lined Tanks

The foregoing considerations apply to unlined tanks. If a reliable flexible lining, capable of stretching and bridging a crack about 0·001 in. wide, is provided, a compressive


b
0·1 d
Asc
d
Ast
0·1 d
fcb = 1,200 lb./sq.in.
fst = 12,000 " "
fct = 270 lb./sq.in.
m = 15
Strength Asc = Ast
Strength Asc = 0
Crack Asc = Ast
Crack Asc = 0
M/bd²
400
300
200
100
0
0
1
2
3
4
100 Ast/bd PER CENT.

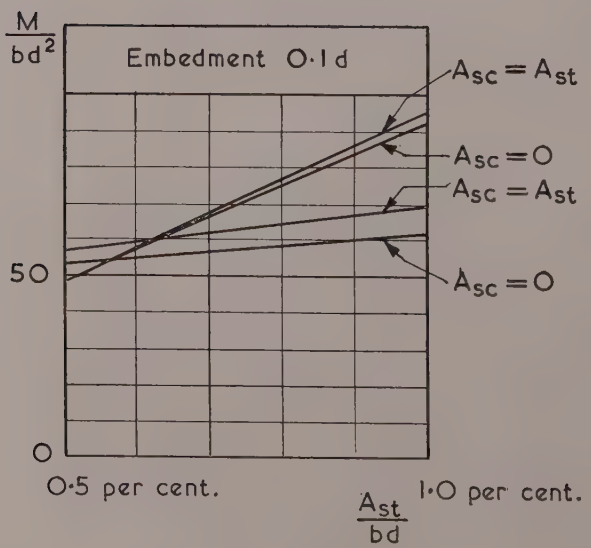

M/bd²
Embedment 0·1 d
Asc = Ast
Asc = 0
Asc = Ast
Asc = 0
50
0
0·5 per cent.
1·0 per cent.
Ast/bd

Fig. A1.4.

stress of 1,200 lb. per sq. in. in the concrete and a tensile stress of 18,000 lb. per sq. in. in the reinforcement could be adopted without reference to the tensile stress in concrete. If d_1 is 0·9, the curves in *Fig.* A1.5 could be used throughout the design.

Members Subjected to Bending Only: Tension at Face Remote from Liquid

Slabs not Thicker than 9 in.—The Code recommends that such slabs should be designed in the same way as slabs with tension at the face in contact with the liquid; therefore the foregoing rules and *Figs.* A1.2 to A1.4 apply.

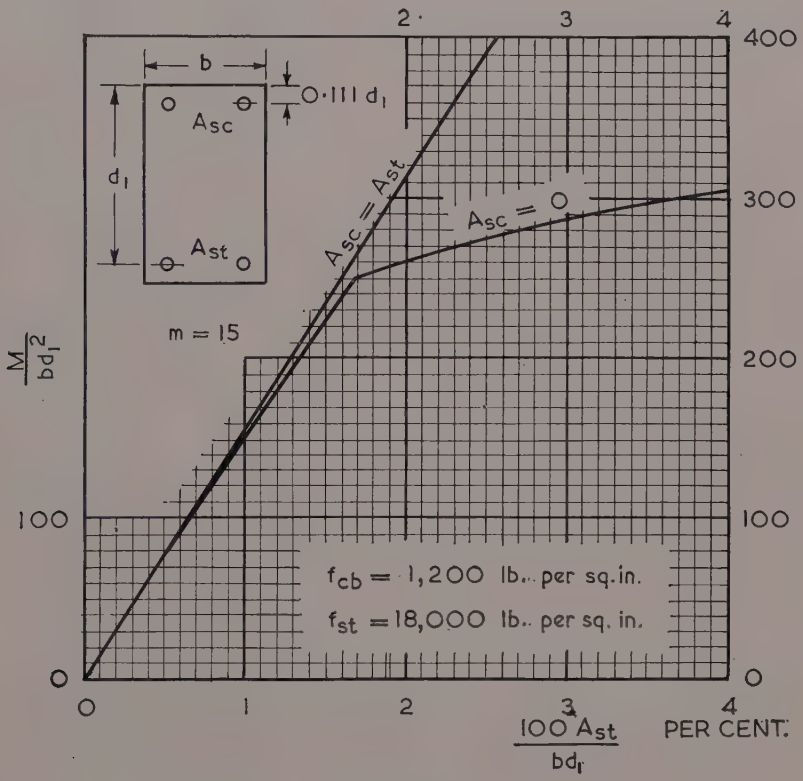


Fig. A1.5.

Slabs Thicker than 9 in. and Ribs of Beams.—In these circumstances no limit is placed on the tensile stress in the concrete, and it is suggested that the tensile stress in the reinforcement be 18,000 lb. per sq. in. With reinforcement in tension only and stresses of 1,200 lb. per sq. in. in the concrete and 18,000 lb. per sq. in. in the reinforcement, $M = 250bd_1^2$, the reinforcement being 1·67 per cent. of the area bd_1. Values of M/bd_1^2, corresponding to stresses not exceeding 1,200 lb. or 18,000 lb. per sq. in., are shown in *Fig.* A1.5 for $d_1/d = 0·9$. These values may be compared with those in *Fig.* A1.4 bearing in mind that this chart is based on the overall depth d while *Fig.* A1.5 is based on the effective depth d_1.

Consider a member reinforced in tension only, the area of reinforcement being 1·5 per cent. of the gross area bd, that is 1·67 per cent. of the effective area bd. For the stresses in *Fig.* A1.4, $M/bd^2 = 135$. For the stresses in *Fig.* A1.5, $M/bd_1^2 = 250$ or $M/bd^2 = 203$, which is 50 per cent. more than the value obtained from *Fig.* A1.4.

If the member has equal reinforcement in tension and compression, say $A_{sc} = A_{st} = 2$ per cent. of the gross area bd (or 2·22 per cent. of bd_1), it is seen from *Fig.* A1.4 that $M/bd^2 = 189$, but from *Fig.* A1.5 that $M/bd_1^2 = 352$ or $M/bd^2 = 284$; this again is about 50 per cent. more than the value obtained from *Fig.* A1.4. In each case the difference is due to the increase of the permissible stress in the reinforcement from 12,000 to 18,000 lb. per sq. in.

If the tension reinforcement is 2·7 per cent. of the gross area bd or 3 per cent. of the effective area bd_1, and there is no compression reinforcement, then from *Fig.* A1.4, $M/bd^2 = 233$, and from *Fig.* A1.5, $M/bd_1^2 = 288$ or $M/bd^2 = 233$. These values are the same, since the resistance of the member is now limited by the compressive strength of the concrete. If, however, compression steel is provided such that $A_{sc} = A_{st} = 2{\cdot}7$ per cent. of bd, that is 3 per cent. of bd_1, M/bd^2 in *Fig.* A1.4 is 253, and M/bd_1^2 in *Fig.* A1.5 is 472 or $M/d^2 = 382$, which is an increase of 50 per cent.

Comments on the Code

Early tanks and reservoirs in reinforced concrete were mostly designed for stresses of 600 and 16,000 lb. per sq. in., with concrete covers of $\frac{1}{2}$ in. in slabs and 1 in. over the main bars in beams and columns. Many of these tanks developed troublesome leaks and much rusting of reinforcement took place. To counteract this, provisions were included in practice that the calculated tension in the wall of a circular tank should not exeed 200 lb. per sq. in. in the concrete and 13,500 lb. per sq. in. in the steel. At the same time the cover to the reinforcement was increased to 1 in. in slabs and $1\frac{1}{2}$ in. over the main bars in beams and columns. With these provisions tanks designed and built by competent engineers were inherently sound but some of those designed by the less competent still developed troublesome cracks and rusting of the reinforcement still occurred. The object of producing the Code was to remedy these defects, but it is not possible to find a sound remedy unless the causes of the trouble are known and understood.

Cracking.—Cracking of concrete may be due to faulty design, faulty specification, faulty construction, hot liquids, or accidents. These causes are examined in the following. Comments on working stresses are given in Appendix II.

Faulty Design.—Faults in design are generally due to failure to understand the limitations of the fundamental assumptions, and failure to grasp the fact that concrete and steel *must* act in unison. It is essential that the student should begin his theory of structures by assuming an ideal elastic material free of all internal accidental stresses and a structure where the members rest on frictionless and incompressible supports. Few structures stand on solid rock and all other foundations compress or settle to some extent, and to some extent unevenly, causing the structure to distort slightly. The stresses caused by this, added to other stresses, may cause slight cracking. Although stresses due to shrinkage of the concrete and changes of temperature may be reduced by good design and specification and sound construction, they are present in all structures. Broad shallow beams may look effective, but if their depth is little more than that of the slabs they ' support ', their deflections may be sufficient completely to upset the calculated bending moments in the slabs. Stopping or overlapping of bars in tension areas always causes local stresses which again are additive to the calculated stress; this was the sole cause of serious cracking in a small water tower in which the bars were stopped exactly in accordance with the calculated bending moments, shearing forces and main tensile forces.

The writer's criticism of the outlook provided by the Code on design is that it makes overprovision for those sections where the calculated moments are a maximum and quite insufficient provision for all sections which may be heavily stressed due to other causes. For small tanks, long bars running right across and hooked at both ends are the best provision against cracking. If this is done, irrespective of where the bars should theoretically be stopped, stresses of 16,000 lb. per sq. in. in bending and 13,500 lb. per sq. in. in direct tension are permissible.

Faulty Specification.—To be effective the Specification must lay down the sequence

of concreting and the maximum intervals between concreting various sections of the work, for example: columns, floor beams, floor slabs, and walls. Cracking in a long battery of gas-purifier boxes resulted entirely from disregard of this requirement; the boxes were supported on high columns which could flex sufficiently to take up shrinkage movement in the long heavy floor slab, but instead of starting from one end and completing each box in turn, the entire long bottom slab was constructed before starting on the walls of the first box; the floor had therefore already experienced an appreciable proportion of its shrinkage, while the walls still had all their shrinkage to come; the difference inevitably caused a series of fine vertical cracks in the walls. Severe and unnecessary restriction of calculated stresses to resist applied loading is useless if unrestricted stresses due to relative shrinkage are allowed to occur. This point is not properly emphasized in the Code.

Wheeled containers of concrete are sometimes run over reinforcement that is already fixed, and bars are bent aside if they are in the way of plant or restrict access. These and other malpractices should be forbidden by the Specification for all types of structure but their avoidance is more important in watertight work. A note to this effect should be included in the Code.

Many cracks in existing tanks have been caused by stripping timber shutters from walls early on a cold morning, the concrete having been cast the day before and being still warm but not strong enough to stand the sudden chilling. This practice should be forbidden, and the Code should say so.

Faulty Construction.—Failure of the contractor to comply with the Specification is dangerous on all work and needs no comment. To what extent the designing engineer should supervise the design of the shuttering and other falsework is a matter of opinion. The writer would not hesitate to design and detail the shuttering and scaffolding if need be, but few engineers would go so far. No opinion (perhaps wisely) is expressed on this point in the Code.

Hot Liquids.—Cracks in tanks holding warm liquids may be greatly reduced by choosing a suitable shape and type. A circular tank with thin walls is much less vulnerable than a square tank with thicker walls. A strong cold wind eddying around the corners of a square tank will chill the corners to many degrees below the middle parts of the sides and the result can be visible horizontal cracks in the corners. This and similar matters are ignored in the Code.

Accident.—This cause is included to complete the list of possibilities, since no Code can deal with the unexpected. An example of what may happen is the following. A watertight basement in water-bearing ballast was perfect except at one spot where water came up alongside a splice bar set in the bottom slab. The most likely cause was that something, perhaps a crane skip, must have hit the upstanding bar just after the concrete had taken its initial set and thereby left a passage for the ingress of the water.

Cover to Reinforcement.—The Code makes the same fundamental mistake as B.S. Code of Practice CP.114 in confusing nominal cover with actual cover.

In *Fig.* A1.6*a* is shown diagrammatically what happens when a light skeleton of column reinforcement has 1-in. nominal cover and *Fig.* A1.6*b* illustrates what happens when it has 2-in. nominal cover, no proper spacers being provided in either case. The rusting of steel in such a column follows a uniform pattern. The surface is usually given a coat of cement wash soon after the shuttering is removed and nothing happens during the first winter. The contractor is therefore exonerated since the period of maintenance may be six or twelve months. After two or three winters small brown patches appear; these are rust marks caused by the ends of the binding wire which actually touched the shuttering. Another winter or two sees these marks developing into circular blown-off patches which reveal the column links. The trouble spreads into long horizontal ' blows ' which eventually reach the main bars and develop into major vertical ' blows ' which remove the corners of the column. If the designer fails to specify and include proper bar spacers in the Bills of Quantities and if no one cares what happens to the spare ends of the binding wire, large nominal covers are useless.

Repairs are always difficult, always expensive and always unsatisfactory. An actual

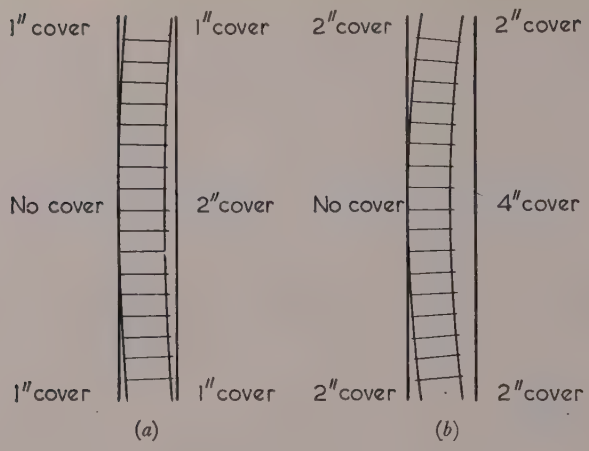


Fig. A1.6.

cover of 1 in. in slabs with bars of less than 1-in. diameter is sufficient. An actual cover of $1\frac{1}{2}$ in. over the main bars in beams, counterforts and columns is also sufficient.

APPENDIX TWO

WORKING STRESSES AND SPECIFICATIONS

BEFORE commencing a design several conditions must be considered to assess reasonable working stresses. These include (1) the materials available, (2) the labour available, (3) the plant available, (4) the quality of supervision, and (5) any particular conditions applying to the proposed work such as the local climate.

MATERIALS.—In many areas in Great Britain good aggregate is not produced. An additional haulage of one mile adds (in 1962) about 6*d.* per ton to the cost. Local aggregate may therefore have to be used with lower stresses in the concrete (or a richer mix), or better and more expensive aggregate obtained from a distance.

LABOUR.—There are obvious limitations of native labour on contracts abroad. British labour may vary with the district and the type of contractor. A contractor who specializes in reinforced concrete has an organization of senior workmen who are (or should be) experienced in the construction of reinforced concrete. A builder attempting his first reinforced concrete work is here at a disadvantage. The designer should estimate the accuracy to which the available labour can reasonably be expected to work, and prepare a design and specification within these limits.

PLANT.—The plant required varies from a large contract with a full complement of new and efficient plant to small work in a remote district for which there may be available only a few items of worn-out plant. A good specification will call for a reasonable minimum amount of plant having regard to the size of the work.

SUPERVISION.—The supervision probably varies more widely and has more effect on the quality and strength of concrete than the two preceding items. At one end of the scale is very large work on which every operation is carried out under independent experienced supervision, and at the other end the very small contract left entirely to a junior leading-hand with no independent supervision.

PARTICULAR AND PERSONAL CONSIDERATIONS.—Apart from the vagaries of climate, everyone connected with a contract has his own fancies or foibles, and to make a reasonably close estimate of the extent to which these should be humoured or opposed is one of the engineer's problems. Variations are too numerous to list, but they often play an important part in the wording of a specification or the detailing of a design. For example, if the only means of placing the concrete is by a chuting plant, then the designer must specify a chutable mix such as 1 : 2 : 3 or 1 : $1\frac{1}{2}$: $2\frac{1}{2}$.

EFFECT OF SIZE OF STRUCTURE.—The type of plant and quality of supervision almost always vary with the size of the work. Control of the quality of materials and the labour employed are also generally better on large works. For large works it is reasonable to draft a detailed specification calling for close control of all operations, and it is possible to adopt a complicated arrangement of reinforcement with high working stresses in the concrete. On many small works detailed technical wording may lead to misunderstanding and mistakes, while a complex arrangement of reinforcement may result in reinforcing bars being left out. For small isolated works it is unwise to use a mix leaner than 1 : $1\frac{1}{2}$: 3, to issue any specification other than a few clearly-worded directions, or to have complicated reinforcement details; a working stress of 750 lb. per square inch should not be exceeded.

EFFECT OF SIZE OF MEMBERS.—In B.S. Code of Practice CP.2007:1960 there is a failure to distinguish between allowable stresses in the concrete in small members and in large members. In a small thin member a single batch of poor concrete will ruin its strength. A large thick slab would suffer little from just one batch of poor concrete. In thin sections the concrete stresses must be based on reasonable minimum cube-strengths, while in heavy construction the average cube-strength is the proper criterion. The writer would use a

1 : 1½ : 3 (as was common before the Code was published) where a 1 : 1·6 : 3·2 mix is recommended in the Code, and in reasonably solid construction would adopt the following working stresses in the concrete:

In direct tension in circular tanks:	200 lb. per sq. in.
In direct compression:	950 ,, ,, ,, ,,
In compression due to bending:	1,250 ,, ,, ,, ,,
In shearing:	115 ,, ,, ,, ,,
Bond stress average:	135 ,, ,, ,, ,,
Shear-bond:	200 ,, ,, ,, ,,

In light construction 80 per cent. of these stresses would be reasonable maxima.

If unhampered by official restrictions, the writer adopts a calculated stress in tension in the steel of 16,000 lb. per sq. in. due to bending and 13,500 lb. per sq. in. in hoop tension. The only sources of trouble encountered have been temperature cracks caused by filling tanks with hot liquid on a cold day.

Lined Tanks.—The adoption of low calculated working stresses is not called for if a waterproof lining (usually asphalt) is provided. It may be argued that a liquid-containing structure is certain to receive its full design load, while an office floor designed to carry a nominal imposed load is unlikely ever to do so. On the other hand it is unlikely, and often impossible, that a tank will ever be overloaded, while there is always the chance that someone may place a heavy safe on a floor not designed to carry it. It therefore seems reasonable to use the same working stresses for both.

Factor of Safety in Construction

Much has been written about factors of safety in completed structures but such discussions are inaccurate and incomplete unless factors of safety in constructional operations are also considered. To the academic expert a 1 : 1·5 : 3 mix is, apparently, always gauged in the proportions of 112·00 lb. of cement to 1·8750 cu. ft. of sand at standard density to 3·7500 cu. ft. of shingle. No method of gauging, manual or mechanical, is perfect. The author has known of two occasions where it was discovered, when the shuttering was removed, that a batch of concrete, containing no cement whatever, had been gauged, mixed and placed. If it is essential that no batch should fall below 1 : 1·6 : 3·2 the specification must call for proportions of at least 1 : 1·5 : 3.

APPENDIX THREE

DR. REISSNER'S METHOD OF CALCULATING RESTRAINT

THE following notes on Dr Reissner's method have been compiled by Mr H. V. Carpenter, A.M.I.C.E.

In the cross-section of the cylindrical tank shown at (*a*) in *Fig.* A3.1, let x be the depth from the top of the tank to the point considered, d the thickness of the wall at a depth x, du the thickness of the wall at its junction with the base, v the radial deflection of the wall in a

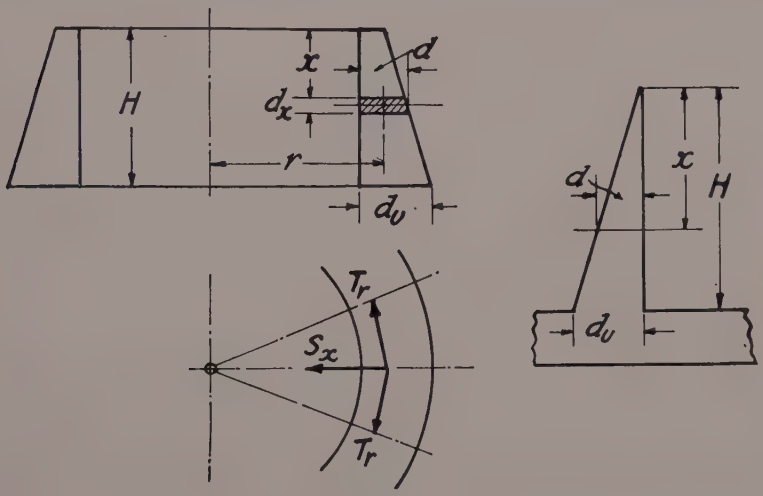


Fig. A3.1.

horizontal direction, u the displacement of the wall in the direction of the x-axis, m Poisson's ratio, and w the weight of the contained liquid per unit of volume.

The strains in the direction of the x-axis and the radius, respectively, due to the horizontal pressure of the contained liquid are

$$\varepsilon_1 = \frac{du}{dx}, \text{ and } \varepsilon_2 = \frac{2\pi(r + v) - 2\pi r}{2\pi r} = \frac{v}{r} \quad . \quad . \quad . \quad . \quad (A)$$

and the stresses corresponding to these strains are

$$T_x = \frac{Ed}{1 - m^2}(\varepsilon_1 + m\varepsilon_2), \text{ and } T_r = \frac{Ed}{1 - m^2}(\varepsilon_2 + m\varepsilon_1) \quad . \quad . \quad (B)$$

The curvature due to the deflection v is

$$i = \frac{d^2v}{dx^2} \text{ (approximately)} \quad . \quad . \quad . \quad . \quad . \quad (C)$$

and the bending moment is

$$M_x = \frac{EI}{(1 - m^2)}\frac{d^2v}{dx^2} = \frac{Ed^3}{12(1 - m^2)}\frac{d^2v}{dx^2} \quad . \quad . \quad . \quad . \quad (D)$$

$$\text{The shearing force } S_x = \frac{dM_x}{dx} \quad . \quad . \quad . \quad . \quad . \quad (E)$$

The conditions of equilibrium expressed in cylindrical co-ordinates are

$$\frac{dT_x}{dx} = 0 \text{ for the forces in the direction of } x \quad . \quad . \quad . \quad (F)$$

$$\frac{dS_x}{dx} + \frac{T_r}{r} = wx \text{ for the forces in the direction of } r \quad . \quad . \quad . \quad (F_1)$$

From equations (B) $T_x = 0$, hence

$$\frac{du}{dx} + \frac{mv}{r} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad (G)$$

Substituting the values of M_x and S_x from equations (D) and (E), and $T_r = Edv/r$ from equation (B), in equation (F_1),

$$\frac{d^2}{dx^2}\left(d^3\frac{d^2v}{dx^2}\right) + \frac{12(1 - m^2)}{r^2}du = 12\frac{(1 - m^2)}{E}wx \quad . \quad . \quad . \quad (H)$$

Walls of Triangular Cross-Section.—Let $\xi = x/H$ and $d = \xi d_u$ (see *Fig.* 3A.1b) so that equation (H) may be written

$$\frac{d^2}{d\xi^2}\left(\xi^3\frac{d^2v}{d\xi^2}\right) + \xi\varkappa v = \lambda\xi \quad . \quad . \quad . \quad . \quad . \quad (J)$$

where

$$\varkappa = \frac{12H^4(1 - m^2)}{r^2{d_u}^2} \quad . \quad . \quad . \quad . \quad . \quad (K)$$

and

$$\lambda = \frac{12H^5(1 - m^2)w}{E{d_u}^3} \quad . \quad . \quad . \quad . \quad . \quad (K_1)$$

The differential equation (J) must satisfy the following boundary conditions.

$$\left.\begin{aligned} &\text{For } \xi = 1{\cdot}0,\ v = 0,\ \frac{dv}{d\xi} = 0 \text{ (fixed end).} \\ &\text{For } \xi = 0,\ M_x = 0,\ S_x = 0 \text{ (free end).} \\ &\text{Or } \xi^3\frac{d^2v}{d\xi^2} = 0,\ \frac{d}{d\xi}\left(\xi^3\frac{d^2v}{d\xi^2}\right) = 0 \end{aligned}\right\} \quad . \quad . \quad . \quad (L)$$

The method of integrating equation (J) adopted by Dr Reissner is based on the theory of linear differential equations with the use of two converging series. The complete integral in open form is

$$v = \frac{\lambda}{\varkappa} + C_1v_1 + C_2v_2 + \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (M)$$

where

$$v_1 = 1 - \frac{\varkappa\xi^2}{2!3!} + \frac{\varkappa^2\xi^4}{4!5!} - \frac{\varkappa^3\xi^6}{6!7!} + \ldots,$$

$$v_2 = \xi\left(1 - \frac{2\varkappa\xi^2}{3!4!} + \frac{2\varkappa^2\xi^4}{5!6!} - \frac{2\varkappa^3\xi^6}{7!8!} + \ldots\right),$$

$$C_1 = -\frac{\lambda}{\varkappa}\frac{v_2'}{v_1v_2' - v_1'v_2}, \text{ and } C_2 = +\frac{\lambda}{\varkappa}\frac{v_2'}{v_1v_2' - v_1'v_2}.$$

In these expressions v_1, v_2, v_1', and v_2' are the values of v_1, v_2 and their differential coefficients for $\xi = 1$.

The deflections have been calculated from these equations for values of $\varkappa$ of 10, 100, 1,000 and 10,000. The bending moment is determined by $\frac{d^2v}{d\xi^2}\frac{\varkappa}{\lambda} = C_2v_1'' + C_2v_2''$; hence

$$\left.\begin{aligned} v_1'' &= -\frac{\varkappa}{3!} + \frac{\varkappa^2\xi^2}{2!5!} - \frac{\varkappa^3\xi^5}{4!7!} + \ldots \\ v_2'' &= 2\xi\left(-\frac{\varkappa}{1!4!} + \frac{\varkappa^2\xi^2}{3!6!} - \frac{\varkappa^3\xi^4}{5!8!} + \ldots\right) \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (N)$$

WALLS OF CONSTANT THICKNESS.—In this case $d = d_u$ and equation (J) becomes

$$\frac{d^4v}{d\xi^4} + \varkappa v = \lambda\xi \quad . \quad . \quad . \quad . \quad . \quad . \quad (P)$$

The boundary conditions are $v = v' = 0$ for $\xi = 1$ (fixed end), and $v'' = v''' = 0$ for $\xi = 0$ (free end). The general solution of equation (P) is

$$v = \frac{\lambda}{\varkappa}\xi + A' \cos n\xi \cosh n\xi + B(\sin n\xi \cosh n\xi + \cos n\xi \sinh n\xi) \quad . \quad . \quad (Q)$$

where $n = \sqrt[4]{\frac{\varkappa}{4}}$,

$$\frac{\varkappa}{\lambda}A' = \frac{-2n \cos n \cosh n - (\sin n \cosh n + \cos n \sinh n)}{n[2 \cos^2 n \cosh^2 n - (\cos^2 n \sinh^2 n - \sin^2 n \cosh^2 n)]},$$

and $$\frac{\varkappa}{\lambda}B = \frac{+n(\cos n \sinh n - \sin n \cosh n) - \cos n \cosh n}{n[2 \cos^2 n \cosh^2 n - (\cos^2 n \sinh^2 n - \sin^2 n \cosh^2 n)]}.$$

For large values of $\varkappa$ and consequently n, as $\sinh n = \frac{1}{2}(e^n - e^{-n}) = \sqrt{\cosh^2 n - 1}$, an approximation may be made, namely,

$$\sinh n = \cosh n = \tfrac{1}{2}e^n \quad . \quad . \quad . \quad . \quad . \quad (R)$$

Introducing the notation $\xi_1 = 1 - \xi$, due to Müller-Breslau,

$$v\frac{\varkappa}{\lambda} = \xi - \frac{\cos n\xi_1 + \frac{n-1}{n}\sin n\xi_1}{\cosh n\xi_1 + \sinh n\xi_1} \quad . \quad . \quad . \quad . \quad (S)$$

and $$\frac{d^2v}{d\xi^2}\frac{\varkappa}{\lambda} = 2n^2\left(\frac{\frac{n-1}{n}\cos n\xi_1 - \sin n\xi_1}{\cosh n\xi_1 + \sinh n\xi_1}\right) \quad . \quad . \quad . \quad . \quad (T)$$

In these equations $\cosh n\xi_1 - \sinh n\xi_1$ can be replaced by $e^{n\xi_1}$. Numerical values of the hyperbolic functions may be obtained from handbooks or from equation (R). As $\log_e e^n = n \log_e e = n$ and $\log_e \frac{1}{2}e^n = n - \log_e 2$, values of e^n may be obtained from a table of natural logarithms.

The ring tension may be found from equations (A) and (B).

$T_r = \left(\frac{Ed}{1-m^2}\right)\frac{v}{r} = \frac{\lambda}{\varkappa}\frac{Ed}{(1-m^2)r}\left(\frac{v\varkappa}{\lambda}\right)$. Substituting in this equation the values obtained from equations (K),

$$T_r = Hrw\left(v\frac{\varkappa}{\lambda}\cdot\frac{d}{d_u}\right) \quad . \quad . \quad . \quad . \quad . \quad (U)$$

From equation (D) the bending moment is

$$M_x = \frac{Ed^3}{12(1-m^2)}\frac{d^2v}{dx^2} = \frac{Ed^3}{12(1-m^2)H^2}\frac{d^2v}{d\xi^2} = \frac{\lambda}{\varkappa}\cdot\frac{Ed_u{}^3}{12(1-m^2)H^2}\left(\frac{\varkappa}{\lambda}\cdot\frac{d^2v}{d\xi^2}\cdot\frac{d^3}{d_u{}^3}\right).$$

Neglecting m^2 and transposing equation (K)

$$M_x = \frac{r^2w}{H}\cdot\frac{I}{A}\left(\frac{\varkappa}{\lambda}\cdot\frac{d^2v}{d\xi^2}\cdot\frac{d^3}{d_u{}^3}\right) \quad . \quad . \quad . \quad . \quad . \quad (V)$$

The term in brackets in equations (U) and (V) is a pure number and may be calculated from equations (M) and (N) for walls of triangular cross-section and from equations (S) and (T) for walls of rectangular cross-section.

The fixing moment at the bottom of a wall of rectangular cross-section is found by putting

$\xi_1 = 0$ or $\xi = 1$ in equation (T), whence $\frac{d^2v}{d\xi^2} \cdot \frac{\varkappa}{\lambda} = 2n(n-1)$. This value inserted in equation (V) gives

$$M_x = \frac{r^2 w}{H} \cdot \frac{I}{A} 2n(n-1) \quad . \quad . \quad . \quad . \quad . \quad (W)$$

Usually it is sufficiently accurate to assume that $I/A = \frac{1}{12} d_u^2$.

In walls of rectangular cross-section the ring tension T_r is zero at the bottom of the tank ($\xi = 1$) and also at the top of the wall ($\xi = 0$) for the larger values of n common in practice. For sufficiently large values of n equation (S) may be differentiated thus, $\cosh n\xi_1 + \sinh n\xi_1 - \cos n\xi_1 - (2n-1)\sin n\xi_1 = 0$, and the maximum value of T_r is where

$$\frac{\cosh n\xi_1 + \sinh \xi_1 - \cos n\xi_1}{\sin n\xi_1} = 2n - 1 \quad . \quad . \quad . \quad . \quad (X)$$

The maximum value of M_x is found for sufficiently large values of n by differentiation of equation (T); thus $-\frac{n-1}{n} \sin n\xi_1 - \cos n\xi_1 - \frac{n-1}{n} \cos n\xi_1 + \sin n\xi_1 = 0$

or

$$\tan n\xi_1 = 2n - 1 \quad . \quad . \quad . \quad . \quad . \quad (Y)$$

For large values of n it is sufficiently accurate to assume $n\xi_1 = \frac{\pi}{2} - \frac{1}{2n-1}$. Hence $\cos n\xi_1 = \frac{1}{2n-1}$ and $\sin n\xi_1 = 1 - \frac{1}{2(2n-1)^2}$, as

$$\sinh n\xi_1 + \cosh n\xi_1 = e^{n\xi_1} = \frac{e^{\frac{1}{2}\pi}}{e^{2n-1}} = e^{\frac{1}{2}\pi}\left[1 - \frac{1}{2n-1} - \frac{1}{2(2n-1)^2}\right].$$

As $e^x = 1 + \frac{x}{1!} + \frac{x^2}{2!} + \ldots$ then $e^{\frac{1}{2n-1}} = 1 + \frac{1}{2n-1} + \frac{1}{2(2n-1)^2}$, and the fraction in equation (T) becomes

$$\frac{\frac{n-1}{n}\cos n\xi_1 - \sin n\xi_1}{\cosh n\xi_1 + \sinh n\xi_1} = \frac{\frac{n-1}{n(2n-1)} - 1 + \frac{1}{2(2n-1)^2}}{e^{\frac{1}{2}\pi}\left(1 - \frac{1}{2n-1} - \frac{1}{2(2n-1)^2}\right)}.$$

As the numerator and the term in brackets of the denominator are approximately unity,

$$\frac{\frac{n-1}{n}\cos n\xi_1 - \sin n\xi_1}{\cosh n\xi_1 + \sinh n\xi_1} = e^{-\frac{1}{2}\pi}\left(1 - \frac{n-1}{n(2n-1)}\right)\left(1 + \frac{1}{2n-1}\right) = e^{-\frac{1}{2}\pi}\left(1 + \frac{1}{n(2n-1)}\right).$$

Hence the maximum value of M_x becomes

$$M_x = -\frac{r^2 w}{H} \cdot \frac{I}{A} \cdot 0{\cdot}414n\left(n + \frac{1}{2n-1}\right) \quad . \quad . \quad . \quad . \quad (Z)$$

Frequently, the top of the wall is connected to the roof of the tank, but this does not materially affect the stresses in the walls because the deflection at the top is nearly zero for tanks of the proportions commonly met with and for walls of rectangular cross-section. For walls of triangular cross-section and tanks with small values of $\varkappa$, ring compression may occur if the deflection at the top of the wall is restrained.

From the foregoing analysis it may be seen that the distribution of load on the wall of a cylindrical tank will depend on $\varkappa$. In *Fig.* A3.2 the load-distribution curves are plotted for values of $\varkappa$ of 10, 100, 1000, and 10,000 for walls of both rectangular and triangular cross-section. The restraining moment in walls of rectangular or triangular cross-section has

been computed from equation (W) for various values of χ. Now $\chi = \frac{12H^4(1 - m^2)}{r^2 d_u^2}$ where m is Poisson's ratio. Following the usual assumption for reinforced concrete that m is equal to zero

$$\chi = \frac{12H^4}{r^2 d_u^2}.$$

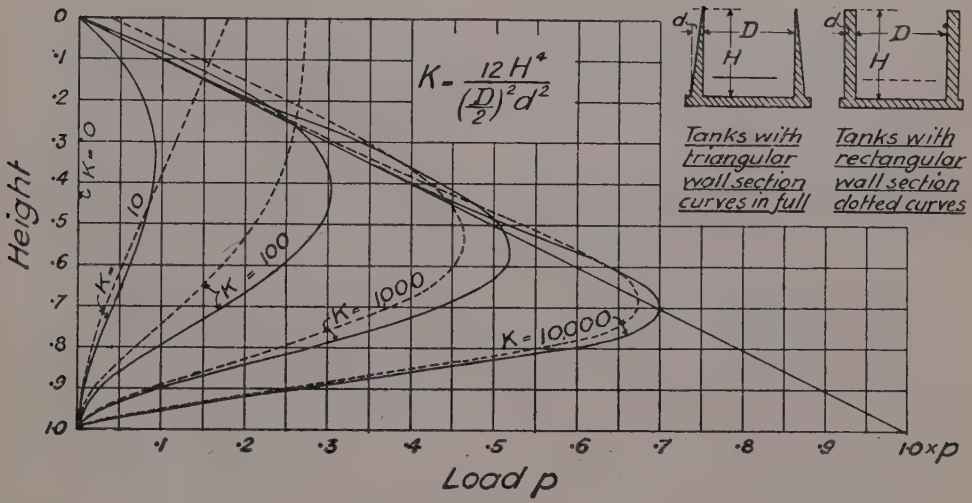


Fig. A3.2.

In Chapter 3 and 4 a criterion of $H/\sqrt{TR}$ is used, where T and R have the same value as d_u and r in Reissner's analysis, so that

$$\chi = 12\left(\frac{H}{\sqrt{TR}}\right)^4.$$

When $\chi = 10$, $\frac{H}{\sqrt{TR}} = \sqrt[4]{\frac{10}{12}} = 0{\cdot}954$,

when $\chi = 100$, $\frac{H}{\sqrt{TR}} = \sqrt[4]{\frac{100}{12}} = 1{\cdot}698$,

etc.

Note.—Equation (W) is strictly only correct for a rectangular cross-section, but it can be applied with slight modification to a triangular cross-section.

ACKNOWLEDGEMENTS

ACKNOWLEDGEMENTS are due to the following engineers, public authorities, contractors and others whose works are illustrated.

Fig. 2.8—F.C. Construction Co., Ltd.

Figs. 21.13, 21.14 and 26.13—Engineer, Mr. J. W. Tomlinson: reinforced concrete design, Messrs. L. G. Mouchel & Partners: contractors, Messrs. Hodges and Porter Ltd.

Figs. 22.4 and 22.5—Reinforced concrete design, Considère Constructions Ltd.

Fig. 22.7—Architects, Messrs. A. S. R. Ley & Sons: contractors, Messrs. Sims & Russell.

Figs. 22.8 to 22.19—Architect, Mr. W. H. Adams: contractors, Messrs. Arundel Ltd.

Figs. 24.6 to 24.9—Mr. B. W. Davies and Messrs. Ritchie & Partners.

Figs. 25.27 and 25.28—Engineer, Mr. Alex. Youill: reinforced concrete design and construction, Messrs. K. Holst & Co., Ltd.

Fig. 25.31—South Western Gas Board and Messrs. A. Monk & Co., Ltd.

Figs. 26.3 and 26.4—Engineer, Mr. H. E. Stilgoe: contractors, Messrs. Wm. Moss & Sons Ltd.

Fig. 26.8—Engineer, Mr. J. Mitchell: contractor, Messrs. Wm. Tawse Ltd.

Figs. 26.9 and 26.10—Engineer, Mr. H. H. Dalymple-Hay: contractors, Messrs. J. Mowlem & Co., Ltd.

Fig. 26.11—Engineer, Mr. W. E. James: reinforced concrete design, Messrs. L. G. Mouchel & Partners: contractors, Welwyn Builders Ltd.

Fig. 26.12—Contractors, Messrs. Lehane, MacKenzie & Shand Ltd.

Fig. 26.14—Designed by Mr. W. Hamilton.

Fig. 26.15—Contractors, Messrs. Mears Bros. (Contractors) Ltd.

Fig. 26.16.—Contractors, Tarmac Civil Engineering Ltd.

Figs. 27.1 and 27.2—Peter Lind & Co., Ltd.

Figs. 27.3 and 27.4—Engineer, Mr. T. H. Longstaff: reinforced concrete design, B.R.C. Engineering Co., Ltd.; contractors, Cumberland Reinforced Concrete Construction Co., Ltd.

Figs. 27.5—Engineers, Mr. C. E. Newton and Mr. J. A. Rodwell: reinforced concrete design and construction, Messrs. Matthews & Mumby Ltd.

Figs. 27.6 and 27.7—Engineer, Mr. W. Plant: reinforced concrete design, B.R.C. Engineering Co., Ltd.: contractors Messrs. Shellabear Price (Contractors) Ltd.

Figs. 27.14 to 27.18—Consulting engineers, Messrs. F. A. Macdonald & Partners.

Figs. 27.19 to 27.22—Consulting engineers, Messrs. John Taylor & Sons assisted by Mr. Leslie Turner: contractors, Messrs. James Miller & Partners Ltd.

Figs. 27.23 to 27.28—Engineer of Cambridge Waterworks Company. Mr. Philip Porteous: consulting engineers, Messrs. Oscar Faber & Partners: contractors, Messrs. William Sindall Ltd.

Figs. 27.29 to 27.32—For Leicester Corporation. Contractors, Messrs. Peter Lind & Co. Ltd.

Fig. 27.34—For Eastbourne Corporation. Roof designed by Mr. William Hamilton.

INDEX

Accidental overload, 13, 376
Adhesion: *see* 'Bond'
Aggregate, 5, 306, 375
Aluminium foil, 99, 172, 299, 362
  ,, lining, 305
  ,, roofs, 78, 349
  ,, wall ties, 316
Arch action in walls, 80, 161
Arches, Segmental, use of as walls, 161, 347
Arrangement of reinforcement, 6, 12, 19, 131, 373

Bar spacers, 3, 258, 373
Basic principles, 1
Beam-and-slab roofs, 258
  ,, walls, 143, 150
Binding wire, 3, 12, 373
Bond, 5, 10, 132, 305, 376
Bottoms: *see* 'Floors'

Caissons, 233, 314
Cantilevered counterfort walls, 107
  ,, slab division wall, 95, 192, 194
  ,, slab walls, 82
  ,, wall on piles, 198
Cement, 4, 176, 311
Channels with sloping bottoms, 101
Circular tanks, Bending in walls of, 21, 279
  ,, ,, Examples of, 10, 232, 300, 331
  ,, ,, in wet ground, 227
  ,, ,, on piles, 196
  ,, ,, Prestressed, 9, 278, 350
  ,, ,, Restraint in, 21 33, 350
  ,, ,, Shallow, 58
  ,, ,, subject to subsidence, 271
  ,, ,, under lateral load, 279
  ,, ,, with interrupted walls, 66
  ,, ,, with sliding joint, 8
  ,, ,, with tapering walls, 49
Circular wall, crippling load of, 358
Code of Practice, 5, 9, 10, 33, 61, 365
Cold liquids, 305, 360
Columns, Precast, 314, 347
  ,, under elevated tanks, 216
  ,, under flat-slab roof, 261
Concrete, Curing, 15, 253, 310
  ,, Materials for, 5, 306
  ,, Placing and compacting, 309
  ,, Proportions of mix, 4, 81, 306, 365, 375–6
  ,, Stresses in, 9, 365
  ,, Transporting, 306
  ,, Watertight, 306
Conical tanks, 236, 243
Construction, 131, 303
Construction joints, 7, 127, 175, 310
Counterforts, 80, 107, 120, 133, 134, 228, 240, 326
Covering existing reservoirs, 347
Cover to reinforcement, 2, 236, 258, 359, 373
Culverts, 147
Curing concrete, 15, 253, 310

Deep rectangular tanks, 221
Division walls, 77, 95, 192, 194
Dowel bars, 177
Drainage, 99, 191, 257, 326

Early tanks, 3, 13, 91, 209
Earth-loaded tanks, 233
Earth pressure, 3, 33, 136, 226
Earthquakes, 226, 286
Economical dimensions, 2, 19
Elastic modulus of concrete, 10, 21, 351
Elevated tanks, 209
Expansion joints, 7, 100, 144, 174, 179, 215, 253, 269, 337

Filter beds, 209
Flat-slab floors, 187, 209, 217, 229
  ,, roofs, 229, 265
  ,, Sunk panel, 186, 261
Floors, Flat slab, 187, 209, 217, 229
  ,, Mesh panel, 184, 209
  ,, Multi-layer, 178
  ,, on compressible ground, 177
  ,, on good ground, 19, 173, 236
  ,, on mass concrete piers, 186
  ,, on patchy ground, 177
  ,, on piles, 187, 209, 215, 226, 243
  ,, on poor ground, 181
  ,, on sleeper walls, 181, 184, 209
  ,, Relief valves in, 225
  ,, Sloping, 101, 236, 243 320, 332
  ,, Splayed slab, 169, 183, 186, 191
  ,, subject to upward pressure, 225
  ,, Suspended, 2, 181, 209
Flotation, 225, 233, 239, 251
Flumes, 147
Formwork, 311

Gasholder tanks, 286
  ,, ,, examples of, 295
Gravity walls, 74
Ground, Drainage of, 99, 191, 257, 326
  ,, Pressure due to, 3, 33, 136, 226
  ,, Waterlogged, 225
Ground water, 74, 225
Gunite, 176, 359, 363

Horizontal loading on tanks, 279
Hot liquids, 305, 373

Imhoff tanks, 248
Industrial liquids, 303
Inclined piles, 198
  ,, walls, 133, 168

Joints between floor and wall, 98, 127
  ,, ,, wall and roof, 48
  ,, Construction: *see* 'Construction joints'
  ,, Expansion: *see* 'Expansion joints'
  ,, for mining subsidence, 275
  ,, Sliding: *see* 'Sliding joints'

Lateral loading on circular tanks, 279
Lattice girders, 274
Liquids, Effect on concrete of industrial, 303

Mass concrete counterfort walls, 80
  ,, ,, footing slabs, 79
  ,, ,, piers, 186
  ,, ,, preliminary layer, 1
  ,, ,, sleeper walls, 181
  ,, ,, walls, 1, 74
Mesh-panel circular roofs, 269
  ,, ,, floors, 184, 209
  ,, ,, roofs, 155, 258
  ,, ,, walls, 153, 158, 159
Mining subsidence, 271
Modular ratio, 8, 351, 366

Narrow tanks, 145

Oil tanks, 303
Ovality of circular tank, 359
Outlet trough, 66, 237, 244

Partial restraint, 35, 41, 57, 66, 356
Petrol tanks, 303
Piled floors, 187, 209, 215, 243
  ,, walls, 196
Pipes through floors, 195
  ,, ,, walls, 251
Precast roofs, 342, 347
Prestressed tanks at low temperatures, 360
  ,, ,, Life of, 9, 278, 359
  ,, ,, Restraint in, 350
Pyramidal tanks, 236

Rapid-hardening cement, 4, 311
Rectangular tanks below ground, 221
  ,, ,, Deep, 222
  ,, ,, Restraint at corners of, 71
  ,, ,, with two compartments, 222
  ,, ,, Small, 154, 230

Reinforcement, Charts for, 367
„ Stresses in, 9
Reissner, Dr., 52, 377
Relief valves, 225
Repairs, 361
Reservoir, Definition of, 1
Restraint at top of wall, 44
„ in large shallow tanks, 58
„ in prestressed tanks, 350
„ in rectangular tanks, 71
„ in tapering walls, 49
„ in uniform walls, 21
„ Partial, 35, 41, 57, 66, 356
„ Reissner's solution, 50
„ under earth pressure, 33
Rocking joint, 214
Roofs, Beam-and-slab, 258
„ Domed, 349
„ Drainage of, 191
„ Flat-slab, 229, 261
„ Mesh-panel, 155, 258
„ Precast, 342, 347
„ Repairs to, 364
„ Small circular, 269

Screw piles, 226
Secondary effects, 123
Segmental arches, 161
Seismic effects, 226, 286
Semicircular tanks, 20
Settlement tanks, 66, 246, 319
Shallow tanks, 58
Shrinkage, 5, 7, 15, 173, 176
Shuttering, 311
Slab-and-girder: *see* 'Beam-and-slab'
Sleeper walls, 181, 209, 220
Sliding forms, 313
Sliding joints, 8, 15, 47, 196, 215, 331
Sloping floor, 1, 101, 236, 243, 320, 332
Sloping wall, 133, 168
Slurry basins, 58, 321
Specifications, 375
Spirally-guided gasholder, 286
Splayed-section slabs, 169, 183, 186, 191
Strutted walls, 133, 342
Sumps, 180
Sunk-panel flat slabs, 187, 261
Suspended flumes, 147
Suspended floors, 2, 181, 289
Swimming pools, 249

Tank, Definition of, 1
Tanks below ground-water level, 225
Tapering walls, 49, 138
Tar tanks, 13
Temperature effects, 5, 7, 15, 46, 136, 143, 173, 211, 216
Tests, 361
Top restraint, 44

Very cold liquids, 305

Walls, Arched, 161
„ Beam-and-slab, 143
„ Cantilevered, 82, 198
„ Counterfort, 107
„ Cracks in, 362
„ Division, 77, 95, 192, 194
„ Drainage below, 99, 191, 257, 326
„ Gravity, 74
„ Joints in: *see* 'Joints'
„ Lateral loading on, 279
„ Mesh-panel, 153, 158, 159
„ of circular tanks, 8, 21, 44, 49, 58, 350
„ „ covered reservoirs, 135, 145
„ „ semicircular tanks, 20
„ „ unusual shape, 339
„ on piles, 37, 196
„ on rock foundations, 133, 144, 155
„ Plain concrete, 74
„ Repairs to, 362
„ spanning vertically, 135, 145
„ Sloping, 133, 168
„ Splayed slab, 169
„ Strutted, 133, 342
„ Tapering, 49, 138
„ under upward pressure, 74, 227
Water-bars, 179
Water pressure under floor, 225, 239, 257
„ „ „ wall, 74, 78, 99, 195, 227
Waterproof lining, 74, 127, 180
Waterproofing additives, 5
Watertightness, 5
Wind effects, 279, 284, 294
Working stresses, 9, 375

Young's modulus: *see* 'Elastic Modulus'

CONCRETE SERIES BOOKS

CONCRETE PUBLICATIONS LIMITED
60 BUCKINGHAM GATE, LONDON S.W.1

The following is a list of titles which have appeared in this Series. For up-to-date details of availability and current prices, please write for a copy of the latest catalogue.

REINFORCED CONCRETE DESIGN

Reinforced Concrete Designer's Handbook. *Reynolds*
Basic Reinforced Concrete Design (2 volumes). *Reynolds*
Explanatory Handbook on the Code of Practice, CP.114. *Scott, Glanville & Thomas*
Examples in the Design of Reinforced Concrete Buildings. *Reynolds*
Structural Lightweight-aggregate Concrete. *Nesbit*
Structural Design Applied to Reinforced Concrete. *Eriksen*
Elementary Guide to Reinforced Concrete. *Lakeman*
Charts for Members Subjected to Biaxial Bending and Thrust. *Pannell*
The Inelastic Space Frame. *Baker*

PRESTRESSED CONCRETE

Introduction to Prestressed Concrete: Two Volumes. *Abeles*
Prestressed Concrete Designer's Handbook. *Abeles & Turner*
Guide to the Code of Practice for Prestressed Concrete, CP.115. *Walley & Bate*
Prestressed Concrete. *Magnel*

STRUCTURAL ANALYSIS & MATHEMATICS

Engineering Mathematics. *Douglas*
Continuous Beam Structures. *Shepley*
Ultimate Load Method of Design. *Baker*
Statically Indeterminate Structures. *Gartner*
Nomograms for the Analysis of Frames. *Rygol*
Analysis of Structures. *Smolira*

CONSTRUCTION & FORMWORK

Concrete Construction. *Reynolds*
Formwork for Concrete Structures. *Wynn & Manning*
Concrete Formwork Designer's Handbook. *Gill*
Concrete Finishes and Decoration. *Childe*
Introduction to Concrete Work. *Childe*
Concrete Construction Made Easy. *Turner & Lakeman*
How to Make Good Concrete. *Walsh*
Concrete Reinforcement Handbook. *Disney*

SPECIAL STRUCTURES

Water Towers, Bunkers, Silos, etc. *Gray & Manning*
Reservoirs and Tanks. *Manning*
Helical Stairs with Fixed Supports. *Cusens & Santathadaporn*
Design of Non-Planar Roofs. *Terrington & Turner*

SPECIAL STRUCTURES *(continued)*

Reinforced Concrete Chimneys. *Taylor & Turner*
Design and Construction of Concrete Roads. *Smith*
Design of Prismatic Structures. *Ashdown*
Arch Design Simplified. *Fairhurst*
Influence Lines for the Fixed Arch. *Erikson*
Reinforced Concrete Bridges. *Legat, Dunn & Fairhurst*

FOUNDATIONS

Design and Construction of Foundations. *Manning*
Deep Foundations and Sheet Piling. *Lee*
Foundation Failures. *Szechy*
Reinforced Concrete Piling and Piled Structures.
Wentworth-Sheilds, Gray & Evans
Raft Foundations. *Baker*

PRECAST CONCRETE

Estimating and Costing Precast Concrete Products and Cast Stone. *Fielder*
Concrete Products and Cast Stone. *Childe*
Concrete Farm Structures. *Pennington*
Moulds for Cast Stone and Concrete Products.
Burren & Gregory

CEMENT

Cement Chemists' and Works Managers' Handbook.
Watson & Craddock
Portland Cement. *Davis*

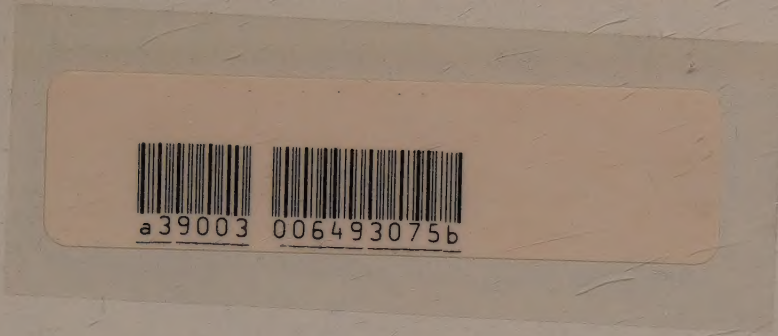
BIBLIOTHÈQUE VANIER
UNIVERSITÉ D'OTTAWA
Échéance

Celui qui rapporte un volume après la dernière date timbrée ci-dessous, devra payer une amende de 10 cents, plus 5 cents pour chaque jour de retard.

VANIER LIBRARY
UNIVERSITY OF OTTAWA
Date due

For failure to return a book on or before the last date stamped below there will be a fine of 10 cents, and an extra charge of 5 cents for each additional day.

APR 21 1974
APR 15 1975
OCT 06 1977
OCT 25 1977
DEC 1 1977
24 AOUT 1981
14 AOUT 1981
12 JUIL. 1982
06 JUIL. 1982
27 JUIN 1983
19 JUIL. 1983
28 JUIL. 1983
25 JUIL. 1983
01 MARS 1984
22 FEV. 1984
MAR 16 1990
07 MARS 1990


a39003 006493075b